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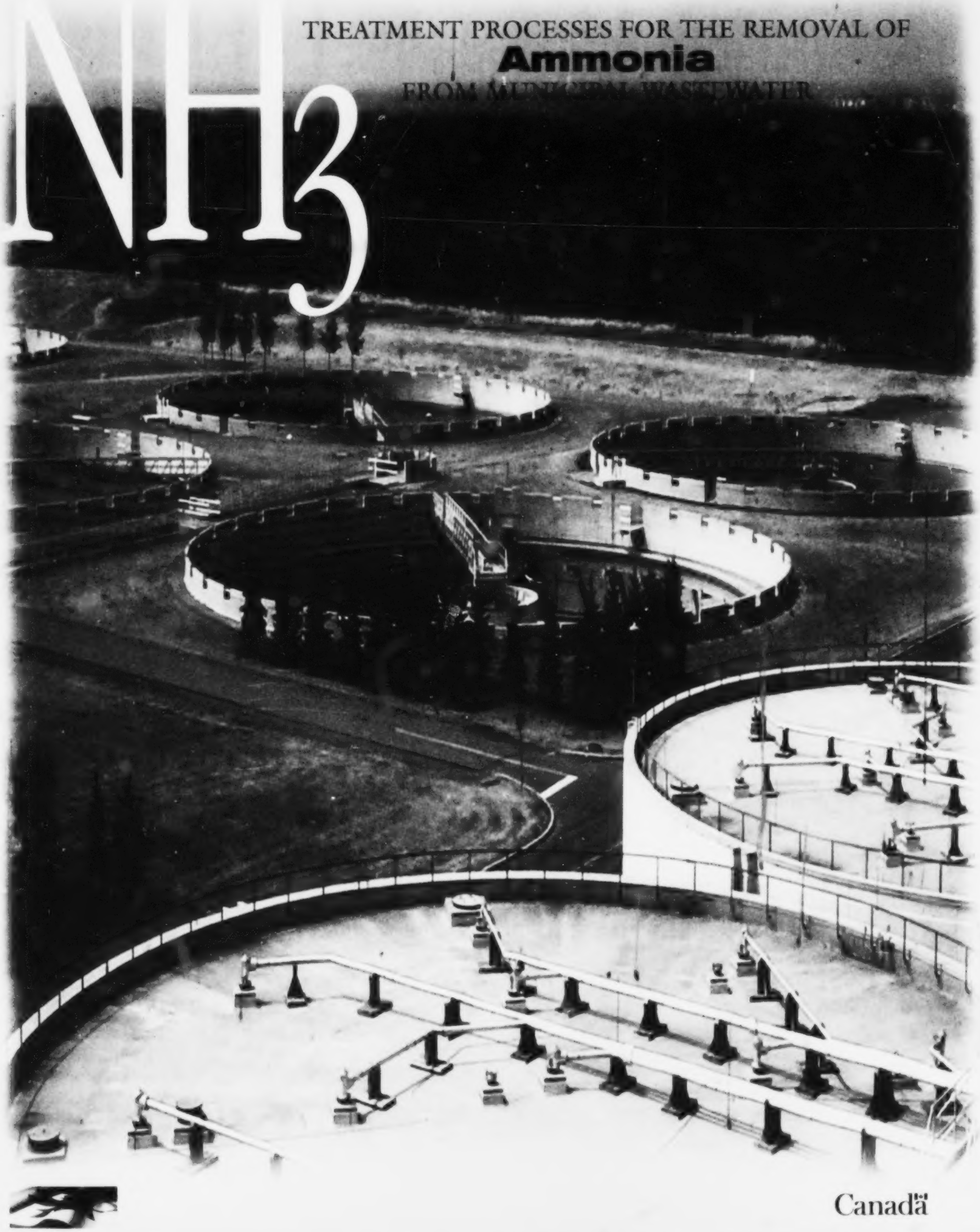
# TREATMENT PROCESSES FOR THE REMOVAL OF
# Ammonia
# FROM MUNICIPAL WASTEWATER



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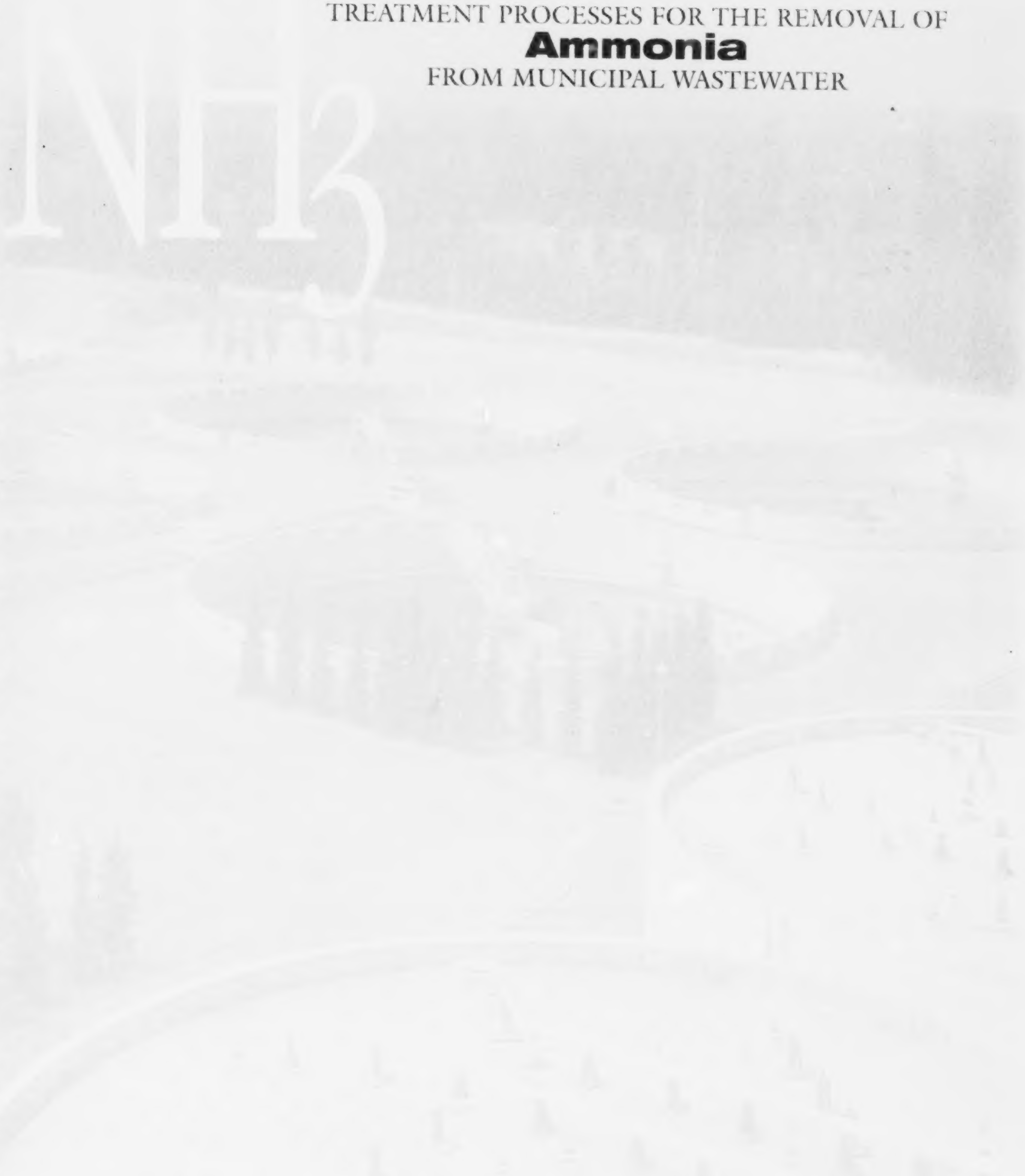
# TREATMENT PROCESSES FOR THE REMOVAL OF **Ammonia** FROM MUNICIPAL WASTEWATER



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# Disclaimer

This report was prepared for Environment Canada. The views and ideas expressed in this report are those of the authors and do not necessarily reflect the views and policies of Environment Canada; nor does the mention of vendors, trade names and commercial products constitute an endorsement or recommendation for use.

The order-of-magnitude-level cost estimations and associated sizes of alternative process options developed for the generic applications described in this report are not intended to be considered as applicable to any particular site that is considering new wastewater treatment facilities or upgrades.

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# List of Acronyms and Abbreviations

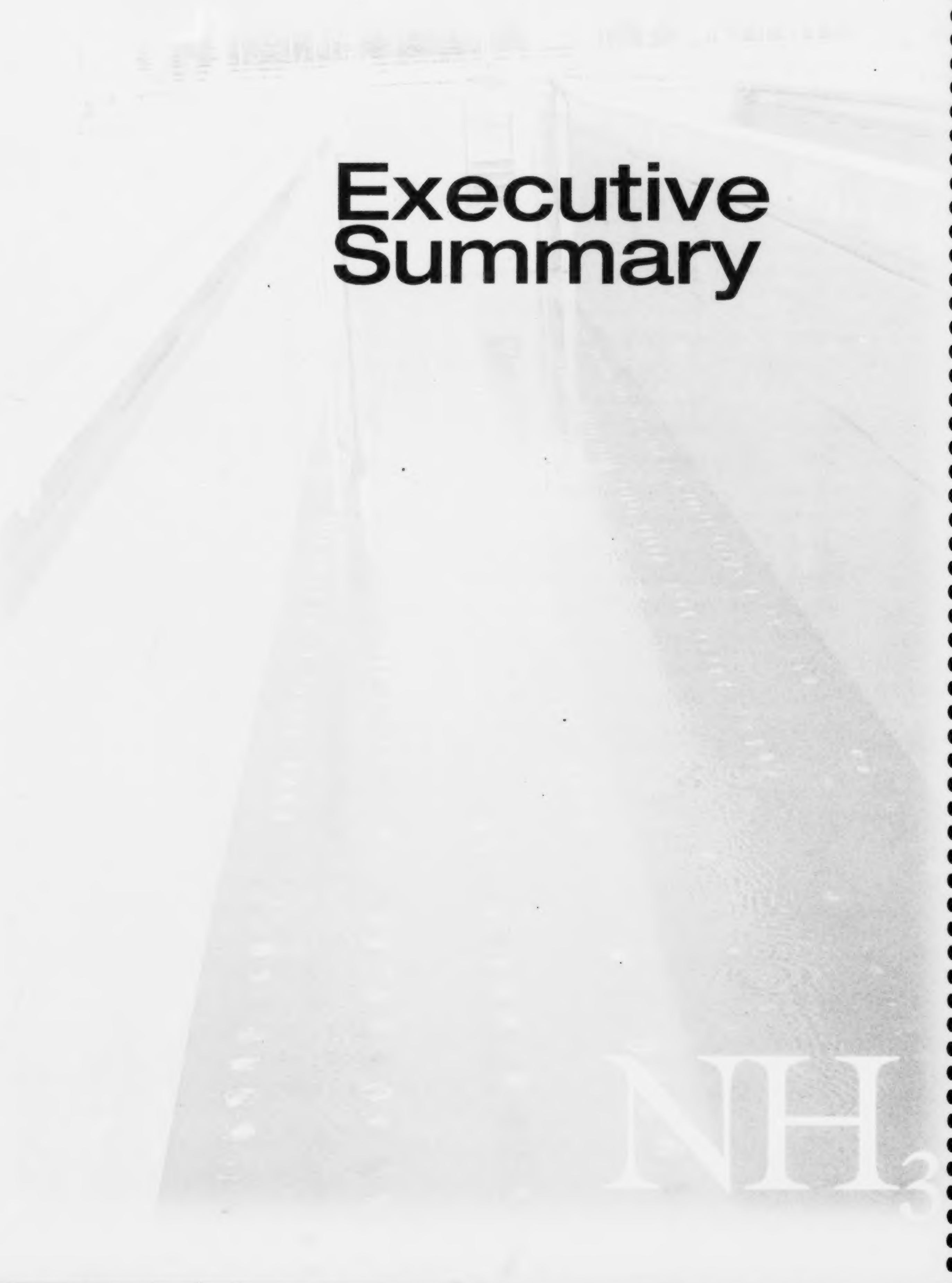
| | |
|---|---|
| $A^2/O$ | anaerobic/anoxic/oxic |
| Ag | silver |
| AgCl | silver chloride |
| APHA | American Public Health Association |
| ASCE | American Society of Civil Engineers |
| ASM | Activated Sludge Model |
| avg. | average |
| AWWA | American Water Works Association |
| BAF | biological aerated filter |
| BCMOE | British Columbia Ministry of Environment |
| BOD | biochemical oxygen demand |
| $BOD_5$ | 5-day carbonaceous or nitrification-inhibited biochemical oxygen demand |
| °C | degree Celsius |
| $CaCO_3$ | alkalinity (expressed as calcium carbonate equivalent) |
| CaO | quicklime |
| $Ca(OH)_2$ | slaked lime |
| CASS | Cyclic Activated Sludge System |
| cBOD | carbonaceous biochemical oxygen demand |
| $cBOD_5$ | 5-day carbonaceous biochemical oxygen demand |
| CCP | Composite Correction Program |
| cfm | cubic feet per minute |
| $CH_3OH$ | methanol |
| $C_5H_7NO_2$ | typical activated sludge biomass composition |
| Cl | chloride ion |
| $Cl_2$ | chlorine |
| $cm^3$ | cubic centimetre |
| $CO_2$ | carbon dioxide |
| COD | chemical oxygen demand |
| CPE | comprehensive performance evaluation |
| CTA | comprehensive technical assistance |
| d | day |
| DO | dissolved oxygen |

| | |
|---|---|
| EBNR | enhanced biological nutrient removal |
| EBPR | enhanced biological phosphorus removal |
| EPA | Environmental Protection Agency (U.S.) |
| °F | degree Fahrenheit |
| F:M | food-to-microorganisms ratio |
| FWS | free water surface constructed wetlands |
| h | hour |
| $H^+$ | free acid |
| $HCO_3$ | bicarbonate |
| $H_2CO_3$ | carbonic acid |
| $HNO_2$ | nitrous acid |
| $H_2O$ | water |
| HRT | hydraulic retention time |
| IAWPRC | International Association on Water Pollution Research and Control |
| IAWQ | International Association on Water Quality |
| I&C | instrumentation and control |
| IFAS | integrated fixed-film activated sludge |
| kg | kilogram |
| kW | kilowatt |
| kWh | kilowatt-hour |
| L | litre |
| lb. | pound |
| $LC_{50}$ | median lethal concentration |
| $M | million dollars |
| m | metre |
| $m^2$ | square metre |
| $m^3$ | cubic metre |
| MBBR | moving-bed biofilm reactor |
| MBR | membrane bioreactor |
| meq | milliequivalent |
| mg | milligram |
| MIGD | million imperial gallons per day |
| mL | millilitre |
| ML | million litres |

| | |
|---|---|
| MLD | million litres per day |
| MLSS | mixed liquor suspended solids |
| MLVSS | mixed liquor volatile suspended solids |
| mm | millimetre |
| MOP | Manual of Practice |
| mV | millivolt |
| MWWTP | municipal wastewater treatment plant |
| N | nitrogen |
| $N_2$ | molecular nitrogen gas |
| N/A | not applicable |
| $Na_2CO_3$ | soda ash |
| NaOH | caustic soda |
| $NH_3$ | un-ionized ammonia |
| $NH_3$-N | un-ionized ammonia-nitrogen |
| $NH_4^+$ | ionized ammonia or ammonium |
| $NH_4^+$-N | ammonium-nitrogen |
| $N_2O$ | nitrous oxide |
| $NO_2$ | nitrite |
| $NO_3$ | nitrate |
| $NO_3$-N | nitrate-nitrogen |
| NOD | nitrogenous oxygen demand |
| $O_2$ | oxygen |
| O&M | operations and maintenance |
| OMOE | Ontario Ministry of the Environment |
| Org-N | organic nitrogen compounds |
| ORP | oxidation-reduction potential |
| OUR | oxygen uptake rate |
| P | phosphorus |
| PCP | pollution control plant |
| POTW | publicly owned treatment works |
| Q | influent |
| QA/QC | quality assurance/quality control |
| RAS | return activated sludge |
| RBC | rotating biological contactor |

| | |
|---|---|
| s | second |
| SBC | submerged biological contactor |
| SBR | sequencing batch reactor |
| SCADA | Supervisory Control and Data Acquisition |
| SK | *Spülkraft*, trickling filter dosing intensity factor |
| sq ft | square feet |
| SRT | solids retention time (or sludge age or mean cell residence time) |
| STAR | Superior Tuning And Reporting |
| TKN | total Kjeldahl nitrogen |
| total $NH_3$-N | total ammonia-nitrogen (i.e., sum of $NH_3$-N and $NH_4^+$-N) |
| TSS | total suspended solids |
| UC | uniformity coefficient |
| UK | United Kingdom |
| USEPA | U.S. Environmental Protection Agency |
| VPISU | Virginia Polytechnic Institute and State University |
| VSB | vegetated submerged bed constructed wetlands |
| VSS | volatile suspended solids |
| WAS | waste activated sludge |
| WEF | Water Environment Federation |
| WERF | Water Environment Research Foundation |
| WTC | Wastewater Technology Centre |
| WTI | Water Technology International Corporation |

# Executive Summary



# Executive Summary

This manual describes techniques to identify and evaluate technological options for application at municipal wastewater treatment plants (MWWTPs) for the removal of total ammonia-nitrogen (the sum of un-ionized ammonia-nitrogen and ammonium-nitrogen) from wastewaters. The report is intended to serve as an information source and decision-support tool to assist municipal engineers, managers and senior process staff at MWWTPs. It discusses the assessment of site-specific treatment requirements and the determination of appropriate approaches for facility upgrading. This can include applications involving the upgrade of existing MWWTPs through process optimization, retrofitting and/or expansion or the selection of suitable new technologies for application as greenfield, replacement or add-on installations.

Detailed descriptions are given of a wide range of processes for ammonia removal, with particular emphasis on various available biological nitrification technologies. These include activated sludge process configurations that can also achieve total nitrogen and/or biological phosphorus removal. Important process-specific technical considerations, design and operational aspects, advantages and limitations, and order-of-magnitude costs for the different processes are provided. Only those treatment processes that have been proven on the full scale or at the large pilot scale in locations with climatic conditions similar to those for Canadian MWWTP applications have been considered. For the purposes of the manual, a target monthly average total ammonia-nitrogen effluent concentration of <5 mg/L has been used as a criterion for process selection and sizing.

The report covers issues such as the need for the generation of pertinent wastewater and treatment process characterization data, the possible requirement for treatability testing and/or pilot plant studies, and available techniques for assessing existing treatment process capacity, capabilities and optimization potential or the requirement for more extensive upgrading. On the basis of process- and site-specific factors and considerations that are outlined and the use of two screening matrix tables that are presented, the municipal engineer will be able to conduct a preliminary identification of suitable upgrade options for enhanced removal of total ammonia-nitrogen. The screening matrices outline specific upgrade categories and various associated measures, which are cross-referenced with the particular level of treatment utilized at an existing facility. Process options for consideration in new greenfield applications are also addressed. This information provides a useful preliminary indication of potential solutions for various cases involving small-, medium- or large-flow applications that require enhanced ammonia removal capabilities. This includes consideration of lagoon process upgrading in cold-climate applications.

With the information provided in this manual, the municipal engineer will obtain the technical background needed to initiate the preliminary phase of an MWWTP upgrade project. This includes the identification of potential remedial options, criteria for the selection of engineering consultants and the ability to work effectively with the consultant(s) to select and design the most suitable process for the particular MWWTP. Actual potential upgrade alternatives for improved ammonia removal will have to be evaluated and demonstrated on a case-by-case basis to meet an MWWTP's effluent objectives, leading to the selection of the most appropriate measure(s).

# Chapter 1
# Introduction

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# Introduction

This manual is an information source document to assist municipal engineers, managers and senior process staff at municipal wastewater treatment plants (MWWTPs) in identifying and evaluating process options available to upgrade existing plants or to install at new plants for the enhanced removal of total ammonia-nitrogen (i.e., total $NH_3$-N). This reference manual provides a basic understanding of nitrification principles, alternative treatment process types and associated designs, operational requirements, technical considerations and associated costs. It focuses on technical methodologies to permit site-specific assessments and determinations of the most appropriate upgrade measure(s), such as optimization, retrofitting, expansion and/or new process installation.

## 1.1 Background

Various approaches are available to upgrade MWWTPs to achieve enhanced removal of total ammonia-nitrogen. The predominant alternative is biological nitrification. Within the broad category of nitrification, there are a number of biological processes that have been successfully applied in Canada and other countries. The most appropriate process for a particular MWWTP application depends on the type and design of the existing liquid treatment process train (e.g., conventional activated sludge, extended aeration, lagoon process, etc.), raw wastewater characteristics, capital and operating costs and other site-specific conditions and considerations (e.g., climate, space availability, age and condition of the facility, etc.).

In this report, the term **total ammonia-nitrogen** (or **total $NH_3$-N**) is used to represent the sum of the nitrogen component of the un-ionized and ionized forms of the compound (i.e., $NH_3$-N plus $NH_4^+$-N), and the term **ammonia** represents $NH_3$ and $NH_4^+$ collectively.

## 1.2 Purpose

Recognizing the complexities in selecting the most appropriate wastewater treatment processes and techniques for removing total ammonia-nitrogen, Environment Canada has funded work on the development of this manual to assist municipalities. With this information source, municipal engineers will have the technical capacity and understanding to initiate the preliminary phase of a treatment plant upgrade project. This includes the preparation of submissions of proposed solutions to municipal council and background information to assist in the selection of engineering consultants for the development of conceptual, preliminary and final plant designs. With the information provided herein, the municipal engineers will be able to work effectively together with their consultants to identify and evaluate appropriate upgrade measures, including MWWTP optimization or the selection and design of the most appropriate process for their MWWTP application.

The manual provides important technical information necessary to identify suitable technological options and to initiate the preliminary phase of plant upgrading to achieve the removal of effluent total ammonia-nitrogen. Consideration is given to the various process types that have been proven at the full scale or large pilot scale at locations with climatic conditions similar to those associated with Canadian MWWTP applications.

The objective of this manual is to assist the municipal engineer in the selection of appropriate options for the enhanced removal of total ammonia-nitrogen from municipal wastewaters. This can include applications involving the upgrade of existing MWWTPs through process optimization, retrofit, expansion or add-on treatment stage or the installation of greenfield plants.

## 1.3 Scope of the Manual

This manual provides municipal engineers, plant managers and senior process staff with an understanding of the fundamental issues concerning ammonia removal to allow them to initiate the preliminary phase of a plant upgrade project. The document also:

- provides a description and assessment of available treatment processes and techniques for the removal of total ammonia-nitrogen from municipal wastewater discharges;
- provides a "screening protocol" for the identification and selection of processes and/or upgrade measures applicable to all size plants, providing comprehensive documentation of approaches available to achieve ammonia removal under a range of scenarios;
- provides evaluation criteria for selecting the most qualified consultant to carry out the work and to allow municipal staff to work effectively with their preferred consultant(s) to select and design suitable processes;
- documents municipal staff roles and resources required in process selection; and
- documents the need for appropriate planning, such as pilot- and full-scale testing and plant optimization.

The manual is intended to serve as a useful reference for municipal engineers by providing pertinent background information on the types of MWWTPs that are utilized in Canada and on the technological options that are available for the enhancement of effluent total ammonia-nitrogen discharge control.

Various scenarios exist for the full-scale application of enhanced ammonia removal techniques. The most appropriate application will be dependent on various site-specific factors that will have to be given consideration. These factors and considerations are discussed in the manual. Process applications can include small-, medium- or large-sized treatment plants. MWWTP applications with service populations of greater than 500 persons were considered in this study.

For the purposes of this manual, a target monthly average effluent total ammonia-nitrogen concentration of <5 mg/L has been used as a criterion for process selection and sizing. This concentration limit should ensure year-round non-acutely lethal effluent discharges (USEPA, 1993; McMaster University and WTI, 1996; NovaTec Consultants Inc., 1996). Nitrifying biological treatment processes would be expected to achieve lower concentrations of total ammonia-nitrogen in effluent discharges during warmer summertime temperature conditions, with somewhat reduced removal efficiencies during colder wintertime periods of operation. By adopting the <5 mg/L effluent total ammonia-nitrogen discharge criterion, in addition to controlling against ammonia toxicity, low concentrations of biochemical oxygen demand (BOD) can be expected from nitrifying biological processes, thus minimizing the influence of BOD on effluent aquatic toxicity (NovaTec Consultants Inc., 1996).

This manual is not intended to provide detailed design information. References are made to other published sources of information on process design aspects, procedures and considerations. Important criteria for the selection of engineering consultants for the development of process designs are provided, and the benefits and requirements for pilot plant testing are discussed.

The manual includes a summary of a number of pertinent case studies describing examples of various full-scale applications and experiences with different types of processes and operational strategies for enhanced ammonia removal. This includes a description of the processes, treatment performance characteristics, operational aspects, cost information and contact persons.

In summary, the manual includes:

- an overview of nitrification process fundamentals;
- a listing and detailed description of ammonia removal technologies in Chapter 5 and Appendix A, with particular emphasis on alternative biological nitrification processes, and including vendor and contact information;
- capital and operations and maintenance (O&M) cost estimates for alternative processes for the defined generic applications;
- matrices to identify process upgrade categories and measures that can be matched with the existing level of treatment at a particular site, to facilitate preliminary decision-making on available remedial options;
- a listing of important process-specific and site-specific factors for consideration;
- an overview of tools to assist in process selection and assessment, such as pilot plant studies, plant audits and process control and troubleshooting options; and
- pertinent case studies of full-scale applications of alternative processes and techniques for ammonia removal.

## 1.4 Approach

Preparation of the manual involved the collection and compilation of pertinent information from numerous sources, such as published reports and technical papers on the state of the art in the removal of total ammonia-nitrogen from municipal wastewaters. Various government representatives at the federal and provincial levels were contacted to obtain up-to-date characterization information on MWWTPs across Canada. Direct technical experience and knowledge of the project team members were relied on for identifying available technological options, technical considerations and full-scale applications and experiences.

Technical supervisors at MWWTPs that were selected as case studies were contacted to obtain site-specific information on the plant design, operational experiences and strategies, and process costs. Numerous vendors of alternative ammonia removal processes, including proprietary technologies, were contacted for specific information on sizing and costs, as well as pertinent full-scale experiences.

A technical steering committee was established to assist in directing the study and to ensure the practicality of the manual. Steering committee members included technical representatives from Environment Canada, the Ontario Ministry of the Environment, the Pollution Control Operations and Services Division of the City of London, Ontario, and technical representatives from the Ontario Clean Water Agency.

## 1.5 Ammonia Removal Processes and Scenarios

Various treatment processes and configurations for ammonia removal are described. For some cases, associated cost estimates were determined. Applications considered in these investigations included greenfield installations and the upgrading of existing secondary treatment processes through optimization, retrofitting, expansion and/or add-on treatment stage(s). Emphasis is given to those technologies and techniques that have been successfully demonstrated at the full scale in municipal wastewater treatment applications.

Consideration has been given to lagoon processes, which, in the case of existing continuous-discharge operations, may require upgrading to provide year-round control of effluent total ammonia-nitrogen to concentrations of <5 mg/L, if necessary, particularly in cold-climate applications. This is an important consideration owing to the large number of lagoon applications in Canada, particularly for smaller municipalities in remote locations.

### 1.5.1 Generic Wastewater Treatment Plants and Scenarios

Two scenarios were defined, representing a small-flow and a medium-flow application, for the identification of suitable ammonia removal process options and corresponding estimates of sizes and respective order-of-magnitude-level capital and O&M costs.

MWWTPs can be divided into the following size categories according to hydraulic processing capacity:

- very small plants = <0.2 million litres per day (MLD) (<0.04 million imperial gallons per day [MIGD]);
- small plants = 0.2–5 MLD (0.1–1 MIGD);
- medium-sized plants = 5–50 MLD (1–11 MIGD); and
- large plants = >50 MLD (>11 MIGD).

The category of very small plants is not considered in this report.

To comparatively assess the size and cost of various alternative processes for ammonia removal, a number of reference conditions and generic treatment scenarios were defined as a basis. The reference conditions used in the study included design average influent flow rates of 1 MLD and 25 MLD, representing small and medium-sized plants. In both cases, the applications investigated involved either new stand-alone ammonia removal treatment processes or add-on processes for the posttreatment of lagoon effluents to ensure consistent year-round nitrification in continuous-discharge mode.

### 1.5.2 Ammonia Removal Process Costs

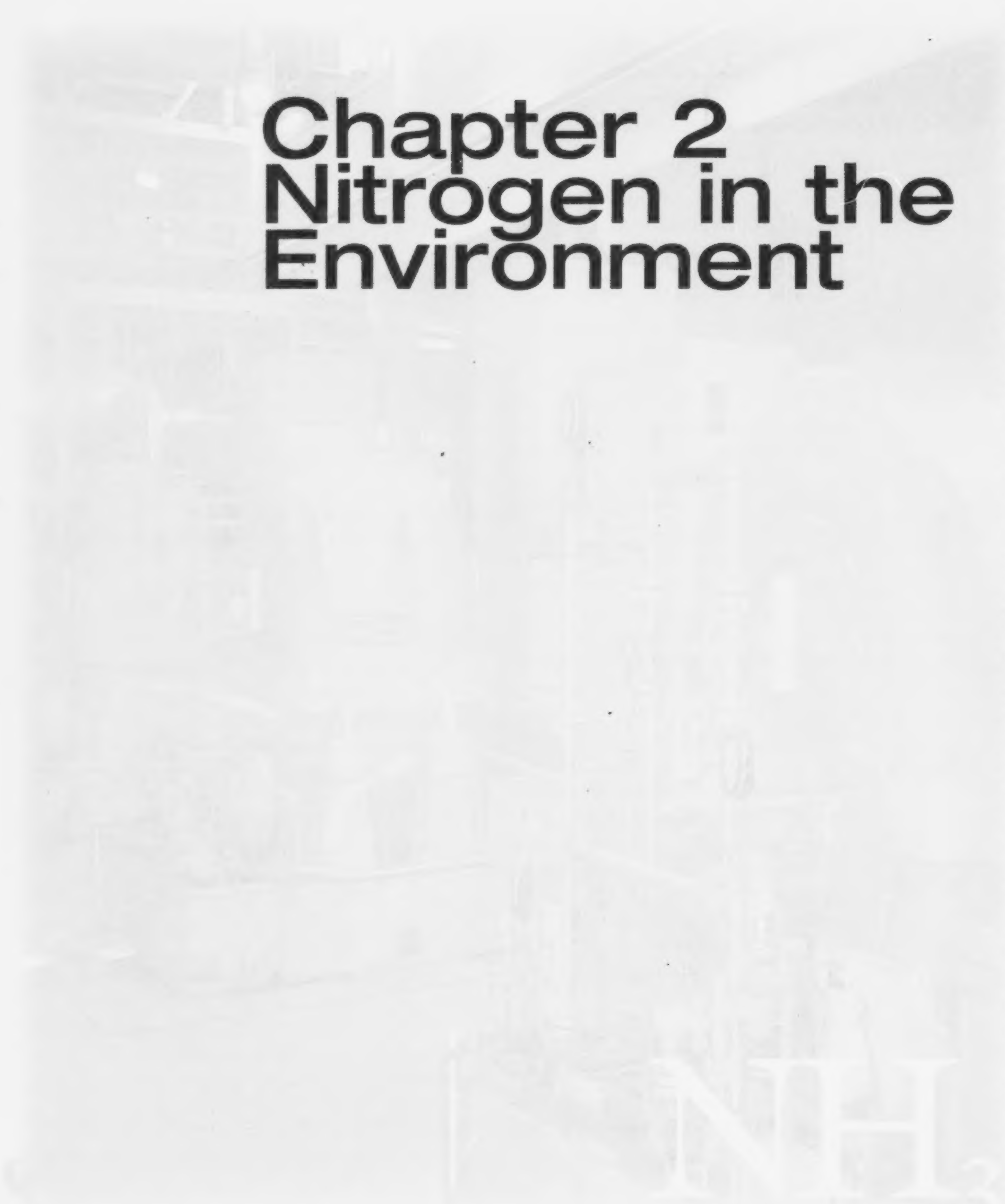
Capital and O&M cost estimates for alternative ammonia removal processes are presented in the report. These are based on published cost-capacity curves for the more traditional biological treatment technologies. The commercially available CapdetWorks™ computer software program by Hydromantis, Inc. was also used to generate order-of-magnitude-level capital and O&M cost estimates of a number of conventional processes for the defined generic MWWTP applications.

For proprietary and less conventional treatment technologies, such as advanced attached-growth processes, vendors were contacted to obtain cost information as well as corresponding process design details for the defined generic applications.

**The reporting of vendor quotations should not be regarded as endorsements of the respective processes, equipment or suppliers.**

In addition to the sizing and costing of new nitrification processes for stand-alone or add-on applications, reported capital cost estimates for various process retrofit options have been cited for comparative evaluation.

# Chapter 2
# Nitrogen in the Environment



# Chapter 2
# Nitrogen in the Environment

This chapter presents an overview of the presence of nitrogen in the environment. The various nitrogen forms and their chemical transformation in the environment are first reviewed in the nitrogen cycle. The major nitrogen sources from natural and human-related activities are identified. Finally, the effects of nitrogen compounds in municipal wastewater discharges on the environment are briefly discussed.

## 2.1 Nitrogen Cycle

The total mass of nitrogen is found in a variety of forms in the atmosphere, surface water and groundwater, soil layers and the organic tissues of organisms. These various forms of nitrogen are transformed through biological and chemical processes and circulate from one environment to the other. This natural nitrogen cycle is being altered by human activities, including the use of minerals and fossil fuels containing nitrogen, industrial chemical nitrogen fixation processes, cultivation of nitrogen-fixing plants and wastewater discharges. The excess of nitrogen developed and fixed by these activities ends up in surface water and groundwater, where undesired effects may result.

Nitrogen exists in the environment under various chemical forms originating from its oxidation states. The five major forms and sources of nitrogen in the environment utilized by microorganisms are summarized in Table 2.1. In this report, the term ammonia represents un-ionized and ionized ammonia collectively, while the term total ammonia-nitrogen represents the sum of ammonia-nitrogen and ammonium-nitrogen.

The various forms of nitrogen and the general biological pathways of nitrogen transformation in the environment are summarized in Figure 2.1. Atmospheric molecular nitrogen ($N_2$) is converted to organic nitrogen compounds (Org-N) by the fixation process.

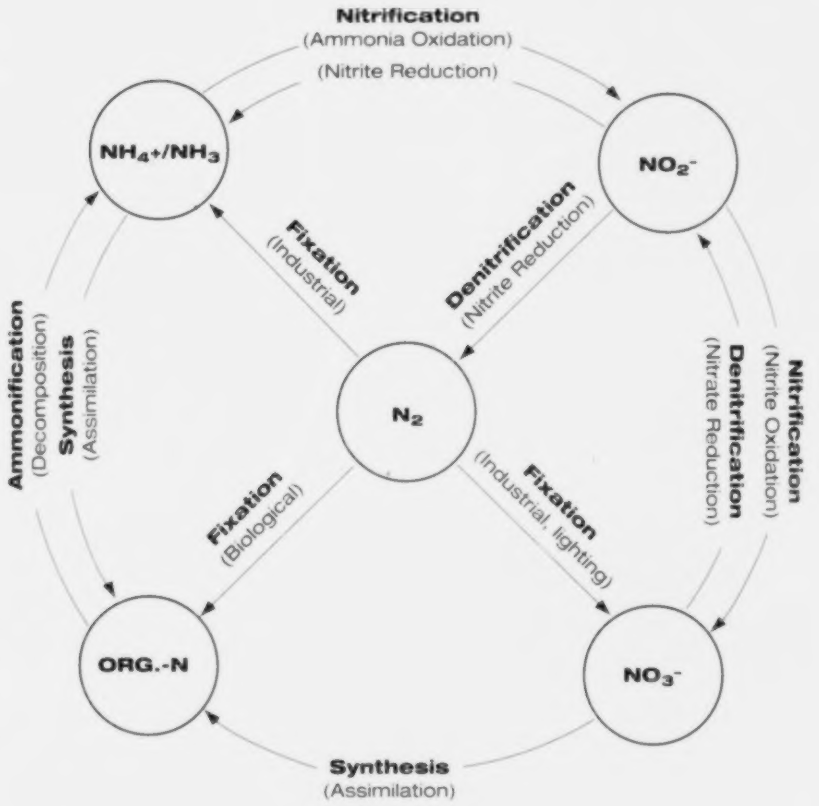
Organic nitrogen compounds are then decomposed by the ammonification process to ammonium and un-ionized ammonia. The nitrification process oxidizes ammonia to nitrite and nitrate. The denitrification process reduces nitrate to nitrite and then returns the nitrogen to the atmosphere as molecular nitrogen.

Nitrate-nitrogen and ammonium-nitrogen may be assimilated by organisms and can be converted to organic nitrogen compounds through the synthesis process. These processes can be carried

**Table 2.1 Major Nitrogen Compounds in the Environment**

| Nitrogen Compound | Formula |
|---|---|
| Un-ionized ammonia or free ammonia | $NH_3$ |
| Ionized ammonia or ammonium | $NH_4^+$ |
| Molecular nitrogen gas | $N_2$ |
| Nitrite | $NO_2^-$ |
| Nitrate | $NO_3^-$ |

**Figure 2.1 Nitrogen Biological Transformation**


Nitrification
(Ammonia Oxidation)
(Nitrite Reduction)
NH4+/NH3
NO2-
Fixation
(Industrial)
Denitrification
(Nitrite Reduction)
Ammonification
(Decomposition)
Synthesis
(Assimilation)
N2
Nitrification
(Nitrite Oxidation)
Denitrification
(Nitrate Reduction)
Fixation
(Biological)
Fixation
(Industrial, lighting)
ORG.-N
NO3-
Synthesis
(Assimilation)

out by various types of microorganisms and are briefly described as follows:

- The **fixation** process involves the transforming of inert molecular nitrogen into organic and inorganic chemical forms (Org-N, $NO_3^-$, $NH_4^+$) that can be further assimilated by organisms through synthesis processes. The fixation process can be accomplished through biological, natural (lightning) and industrial mechanisms, with the biological process being the predominant mechanism.
- The **ammonification** process involves the decomposition of the organic nitrogen compounds from tissues and fecal matter of living and dead organisms to ammonia forms ($NH_4^+$/$NH_3$).
- The **nitrification** process is the sequential biological oxidation of ammonia to nitrite and nitrite to nitrate. This process involves strict aerobic microorganisms using nitrogen compounds for energy production and an inorganic carbon source for cell synthesis. The carbon source is derived from carbon dioxide ($CO_2$) present in the environment. Two bacterial genera are associated with this

two-step process: *Nitrosomonas*, oxidizing ammonium to nitrite, and *Nitrobacter*, oxidizing nitrite to nitrate:

$$NH_4^+ + O_2 \xrightarrow{\textbf{\textit{Nitrosomonas}}} NO_2^- + O_2 \xrightarrow{\textbf{\textit{Nitrobacter}}} NO_3^-$$

- The **denitrification** process is the biological reduction of nitrate to molecular nitrogen. This process occurs in an anoxic environment characterized by the presence of nitrates and the absence of dissolved oxygen (DO). Denitrification is achieved by facultative microorganisms that can evolve in anoxic and aerobic environments. An organic carbon source is required by these heterotrophic microorganisms. The facultative microorganisms first reduce nitrate to nitrite and then reduce nitrite to molecular nitrogen.

$$NO_3^- + \underset{\text{carbon}}{\text{organic}} \longrightarrow NO_2^- + \underset{\text{carbon}}{\text{organic}} \longrightarrow N_2 + CO_2$$

- The **synthesis** process involves the assimilation of ammonium and nitrate by plants and microorganisms to produce organic nitrogen compounds such as amino acids and proteins that can be further assimilated by animals to produce their own proteins and organic tissues.

The biological processes identified as synthesis, ammonification, nitrification and denitrification are the mechanisms involved in municipal wastewater treatment for removal of total ammonia-nitrogen and for total nitrogen control. These reactions are affected by environmental conditions such as temperature, pH, aerobic and anoxic conditions, alkalinity and availability of substrate. The nitrification and denitrification biological processes are described in Chapter 4.

The major effects of nitrogen compounds discharged to the environment are related to surface water characteristics. The nitrogen cycle schematic applied to surface water is shown in Figure 2.2. Various nitrogen forms originating from wastewater discharges (municipal and industrial), precipitation and dustfall, surface runoff (including fertilizers) and atmospheric molecular nitrogen fixation can enter surface water. The nitrogen compounds in surface water can be transformed through synthesis into organic nitrogen. The ammonification of organic nitrogen compounds into ammonia forms can lead to subsequent nitrification and denitrification reactions. These natural biological processes in surface water may, however, be increased by an excess of nitrogen discharge and create negative impacts on the aquatic environment. Total ammonia causes an oxygen demand and, in high concentrations, can be toxic to aquatic organisms.

## 2.2 Nitrogen Sources

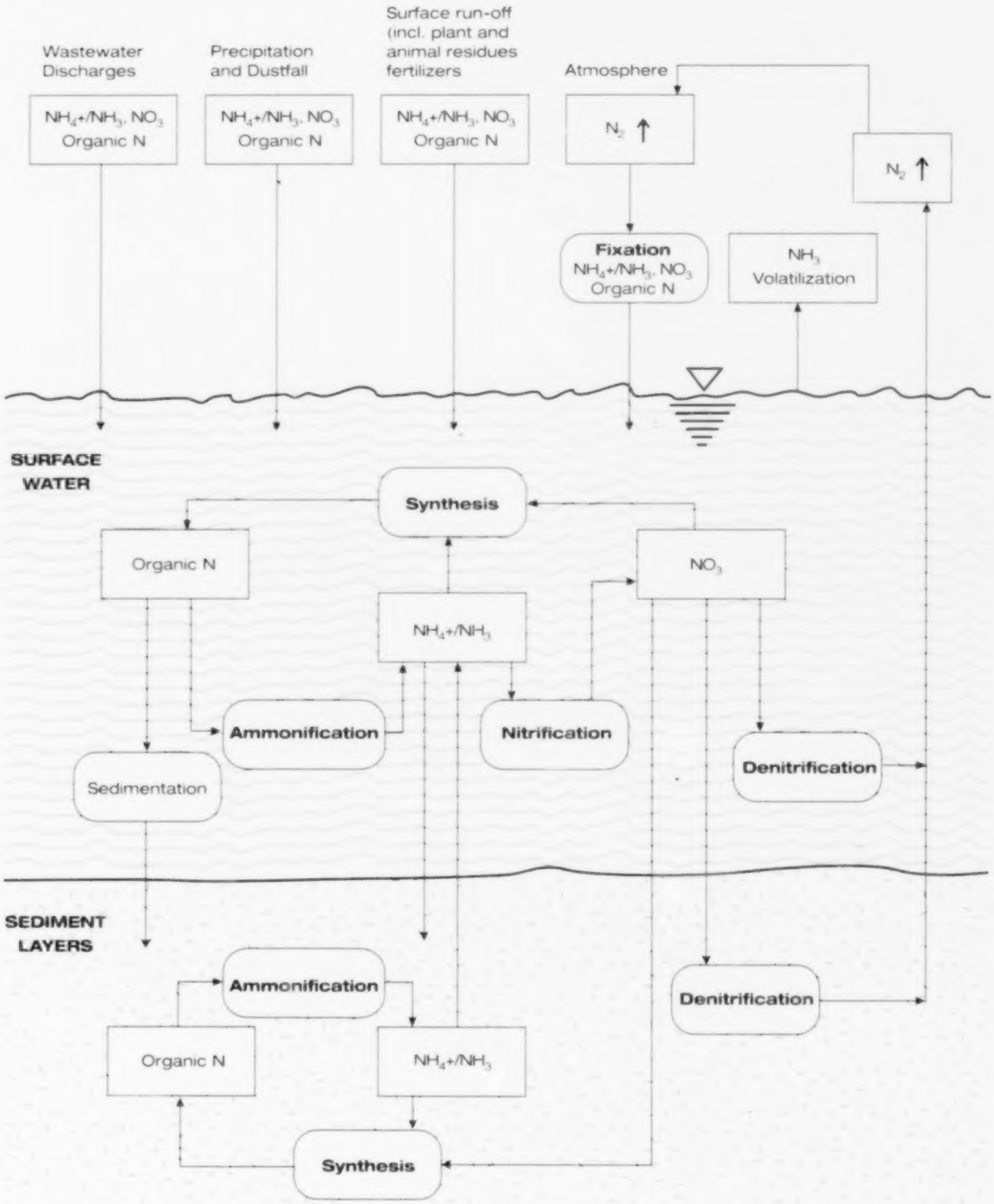
Identification of the various nitrogen sources provides an understanding of the contribution by municipal wastewater discharges to the impacts of excess nitrogen compounds in the environment. An evaluation of the sources of nitrogen being discharged to a receiving body of water helps to identify the most significant sources and to determine the level and type of nitrogen control treatment necessary for those discharges with respect to water quality requirements.

The nitrogen compounds entering surface waters derive from either natural sources or human activities. This section provides a brief overview of these major sources and activities.

### 2.2.1 Natural Sources

The natural sources of nitrogen include atmospheric precipitation, dustfall, surface runoff and biological fixation. The atmospheric fixation of molecular nitrogen through lightning and wind-blown dust contributes to nitrogen compounds contained in precipitation and dustfall. Surface

## Figure 2.2 Surface Water Nitrogen Cycle (adapted from US EPA, 1993)


Wastewater Discharges
$NH_4$+/$NH_3$, $NO_3$ Organic N
Precipitation and Dustfall
$NH_4$+/$NH_3$, $NO_3$ Organic N
Surface run-off (incl. plant and animal residues fertilizers
$NH_4$+/$NH_3$, $NO_3$ Organic N
Atmosphere
$N_2$ ↑
$N_2$ ↑
Fixation
$NH_4$+/$NH_3$, $NO_3$ Organic N
$NH_3$ Volatilization
SURFACE WATER
Synthesis
Organic N
$NO_3$
$NH_4$+/$NH_3$
Ammonification
Nitrification
Denitrification
Sedimentation
SEDIMENT LAYERS
Ammonification
Denitrification
Organic N
$NH_4$+/$NH_3$
Synthesis

runoff from non-urban and non-fertilized areas may contain quantities of nitrogen that vary according to the erosive action on the soils and the volume of precipitation. The biological fixation of atmospheric nitrogen incorporates organic nitrogen compounds in surface water and land environments through organic tissues of plants and animals. These natural sources may, however, be difficult to quantify, as they are influenced by human activities and industrial discharges of nitrogen compounds into the environment.

### 2.2.2 Human Sources

Human sources of nitrogen include inputs such as domestic sewage, industrial wastewaters, landfill leachates and surface runoff. Untreated domestic wastewater contains organic nitrogen and ammonia that originate from human fecal matter. Organic nitrogen comes from proteins, amines, amino acids and other cellular constituents that are excreted in fecal matter. Ammonium originates from bacterial mineralization of proteins and urea.

Treated domestic wastewater contains various levels of nitrogen, depending on the type and degree of treatment. Typically, biological treatment systems reduce the amount of total nitrogen by microorganism cell synthesis and suspended solids removal. The majority of the total ammonia-nitrogen content in the wastewater passes through the treatment system unless an advanced ammonia or total nitrogen removal process is operating. MWWTP sludge processing (e.g., thickening, anaerobic digestion and dewatering processes) contributes to nitrogen recirculation to the front end of the secondary treatment plant. Septage disposal to the treatment plant influent may add a significant amount of nitrogen. Nitrogen recirculation and septage hauled must be taken into account in a nutrient removal treatment plant design.

Industrial wastewaters contribute to nitrogen discharges. Typical industries discharging nitrogen-laden wastewater include, among others, chemical manufacturing, mining and metals processing, rendering plants, food processing, dairy industries and petroleum refineries. Leachates from municipal and industrial solid waste landfills typically exhibit a high degree of variability in terms of leachate strength according to the landfill age and may contain a significant amount of nitrogen. Surface runoff in the urban environment may contain significant levels of nitrogen from industrial and construction suspended solids and stormwater runoff. Surface runoff from agricultural land with fertilizer application may contribute to nitrogen discharges in surface streams and in the groundwater draining the cultivated land areas. Surface runoff may contain atmospheric nitrogen deposition from inorganic nitrogen solubilized in rainwater and particulate nitrogen settling by gravity. Leakage originating from sanitary sewers, industrial tankage and septic fields can represent a contribution to groundwater and stormwater nitrogen content.

## 2.3 Effects of Nitrogen in Municipal Wastewater Discharges

Municipal wastewater discharges can cause excessive nitrogen accumulation in surface water and groundwater. Severe effects on the environment can be attributed to high nitrogen levels. These effects include ammonia toxicity, eutrophication, DO depletion in receiving waters and public health impact.

### 2.3.1 Ammonia Toxicity

Un-ionized ammonia is recognized as a nitrogenous compound responsible for toxicity to aquatic life. Dissolved total ammonia in water exists in equilibrium between two forms, ammonium and un-ionized ammonia:

$$NH_4^+ \rightleftarrows NH_3 + H^+$$

This total ammonia equilibrium is directly related to pH, temperature and total ammonia concentration. The percentage of un-ionized ammonia in the total ammonia load may be determined by the following formula:

$$\text{\% un-ionized ammonia form} = \frac{1}{1 + 10^{(pKa - pH)}} \times 100\%$$

where:

- Ka = ionization constant
- $pKa = 0.09 + \frac{2730}{273 + \text{Temp. } (^\circ C)}$

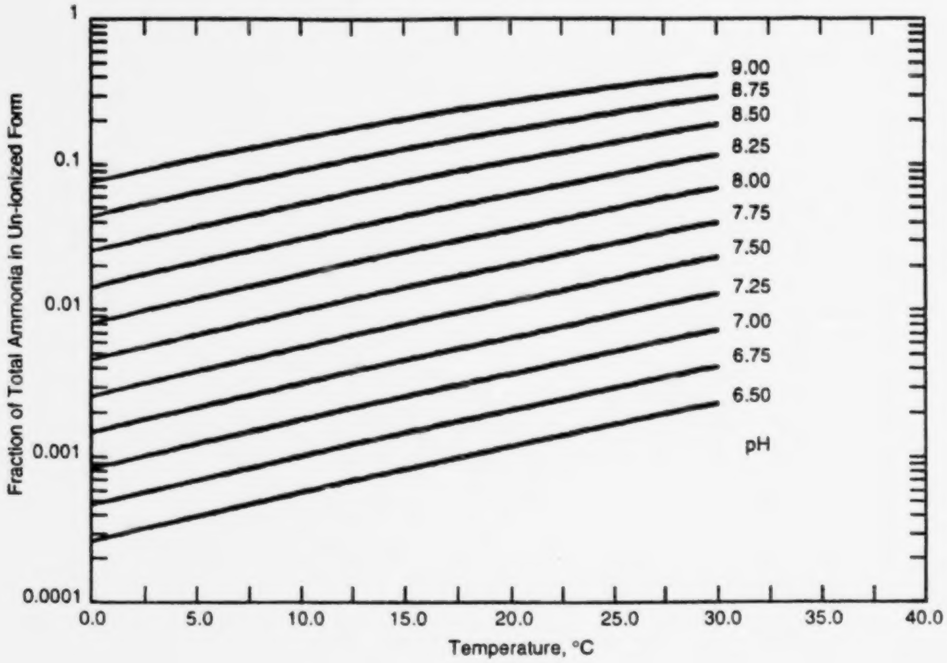
From this equation, it can be seen that at higher pH and temperature levels, the equilibrium balance is driven to a higher concentration of un-ionized ammonia form. This relationship is presented graphically in Figure 2.3.

Toxicity of dissolved total ammonia is mainly attributable to the un-ionized ammonia form, while ammonium is considered significantly less toxic to aquatic life. At sufficient concentrations, un-ionized ammonia can be toxic to fish, as it is lipid soluble and can pass through fish membranes and enter the bloodstream. Acute toxicity is manifested as neurological disorders and gill membrane thickening. Indications of chronic toxicity include a reduction in oxygen diffusion capacity. Both toxicities can result in death to organisms. Un-ionized ammonia is reported to cause acute toxicity in many fish species at concentrations higher than 0.2 mg/L (WEF, 1998). Other reported information indicates that un-ionized ammonia concentrations of greater than 0.1 mg/L may be considered to be acutely lethal (OMOE, 1992). The latter concentration has been used as an effluent limit for some sewage treatment plants in Ontario that discharge into sensitive receiving waters. The U.S. Environmental Protection Agency (EPA) recommends a water quality criterion of 0.02 mg/L for un-ionized ammonia-nitrogen. This criterion incorporates a safety factor of 0.10 for the protection of freshwater aquatic life (Ruffier *et al.*, 1981). The U.S. EPA criteria for ambient water quality provide both maximum total ammonia ($NH_4^+$ and $NH_3$) and un-ionized ammonia toxic criteria levels as a function of pH and temperature (USEPA, 1993; WEF, 1998).

Most freshwater systems have seasonal fluctuations in ambient pH and temperature, with maximum pH in the late spring, summer and fall months, based on available data from southern Ontario. This coincides with higher ambient water temperature. Thus, the end-of-pipe pH may be significantly lower than the ambient water pH. Since the equilibrium between ionized and un-ionized ammonia is highly pH dependent, the concentration of un-ionized ammonia (i.e., toxic form) may be more critical at the pH and temperature conditions at the edge of the initial dilution zone than at the end-of-pipe location. For example, natural stream pH may rise to between 8.5 and 9.0 standard units at the summer peak, corresponding to the highest ambient pH. Thus, effective mixing and initial dilution may be critical to avoid widespread deleterious effects.

Test procedures to determine the toxicity of a given effluent are available. Such tests include 96-hour $LC_{50}$ rainbow trout acute lethality (Environment Canada, 1990a, 1990b), 48-hour $LC_{50}$ *Daphnia magna* acute lethality (Environment Canada, 1990c, 1990d), 21-day *Ceriodaphnia dubia* sublethal toxicity (Environment Canada, 1992a) and 7-day fathead minnow sublethal toxicity (Environment Canada, 1992b) measurements. Additional information related to toxicity testing is provided in USEPA (1994) and Environment Canada (1999).

**Figure 2.3 Temperature and pH Effects on Un-ionized Ammonia (from US EPA, 1993)**



### *2.3.2 Eutrophication*

Eutrophication is the increase in biological productivity occurring in a body of water. Total ammonia-nitrogen discharges to the receiving water may act as a nutrient support and stimulate the growth of aquatic plants and algae. In streams and rivers, nutrients promote the growth of phytoplankton in deep waters and aquatic plants in shallow waters and shorelines. The flowing system and water level fluctuation of deep waters tend to reduce the negative impacts of eutrophication. Aquatic plant growth in shallow waters may, however, provoke greater deterioration of the water quality and create odour problems from plant decomposition and concomitant DO reduction.

In lakes and ponds, eutrophication is a major concern because the nutrients entering tend to accumulate. In these environments, nutrients can promote the growth of aquatic plants and stimulate algal growth, typically referred to as blooms. Algae and aquatic plants whose growth is supported by an excess of nutrients ultimately die off and settle to the bottom of the lake. The decomposition of plants and algae consumes oxygen, which may result in a DO depletion in the water body. The decomposition releases nutrients such as ammonium and phosphates, which are then recycled in the water.

The conditions required for plant and algal growth include the presence of ammonium and phosphate as nutrients, carbon dioxide and light. Nitrogen and phosphorus appear as the two key factors to control eutrophication. Studies have shown that algal growth becomes limiting at an inorganic nitrogen concentration ($NH_4^+$-N and $NO_3$-N) of below 0.05 mg/L (WEF, 1998). However, blue-green algae are able to utilize molecular nitrogen ($N_2$) as a nutrient for their growth. In a situation where nitrogen discharge is controlled but sufficient phosphorus is available, blue-green algae are still able to grow. Death and decomposition of the blue-green algae result in the release of ammonium as a nutrient in water. Removal of nitrogen from wastewater is required when the volume of the receiving water is not sufficient to dilute the discharge of nitrogen.

Phosphorus is considered a growth-limiting factor for algae at concentrations lower than 0.005 mg/L (WEF, 1998). In this sense, when nutrient control is required, phosphorus must be removed, as blue-green algae can grow on molecular nitrogen.

Considering that eutrophication is a water basin issue, it is important to identify the different sources of nitrogen to provide a clear understanding of the wastewater treatment plant contribution to the total nitrogen load in the receiving water.

### 2.3.3 Dissolved Oxygen Reduction

A major impact of ammonia discharge in surface water is the depletion of DO due to in-stream nitrification, which creates an additional oxygen demand. Nitrification is likely to occur in receiving water that provides a favourable environment for nitrifying bacteria. High-rate nitrification is typically related to shallow water, while low-rate nitrification occurs in deep water. In shallow water, sufficient time may be provided for the nitrifiers to grow as attached biomass. The nitrifying population may take advantage of a well-oxygenated stream and favourable temperature to carry on nitrification activity. In the case of shallow water, the volume of water in contact with the attached nitrifying biomass is high, which explains the high rate of nitrification observed in shallow water. According to the oxygenation and dilution capacities of the receiving water, nitrification may occur without any noticeable effect on the DO concentration if stream reaeration exceeds the nitrogenous oxygen demand (NOD). In cases where the NOD exceeds the stream natural reaeration capacity, a significant DO reduction may occur. This reduction may occur on a diurnal basis in cases where in-stream plants produce sufficient oxygen during the day to compensate for the NOD, while at night the respiration activities of the plants along with nitrification can provoke a drastic drop in the DO concentration.

In deep water, only the surface layer is sufficiently oxygenated and warm enough to support the nitrification process. Compared with shallow water, the nitrifying biomass in contact with the volume of water is much lower, which explains the low nitrification rates observed in deep water. Thus, DO reduction has a lower impact in deep water than in shallow water.

In cases where the total ammonia-nitrogen loading of the wastewater discharge is identified as a major contributing factor to the DO depletion in the receiving body of water, its removal by the MWWTP is recommended.

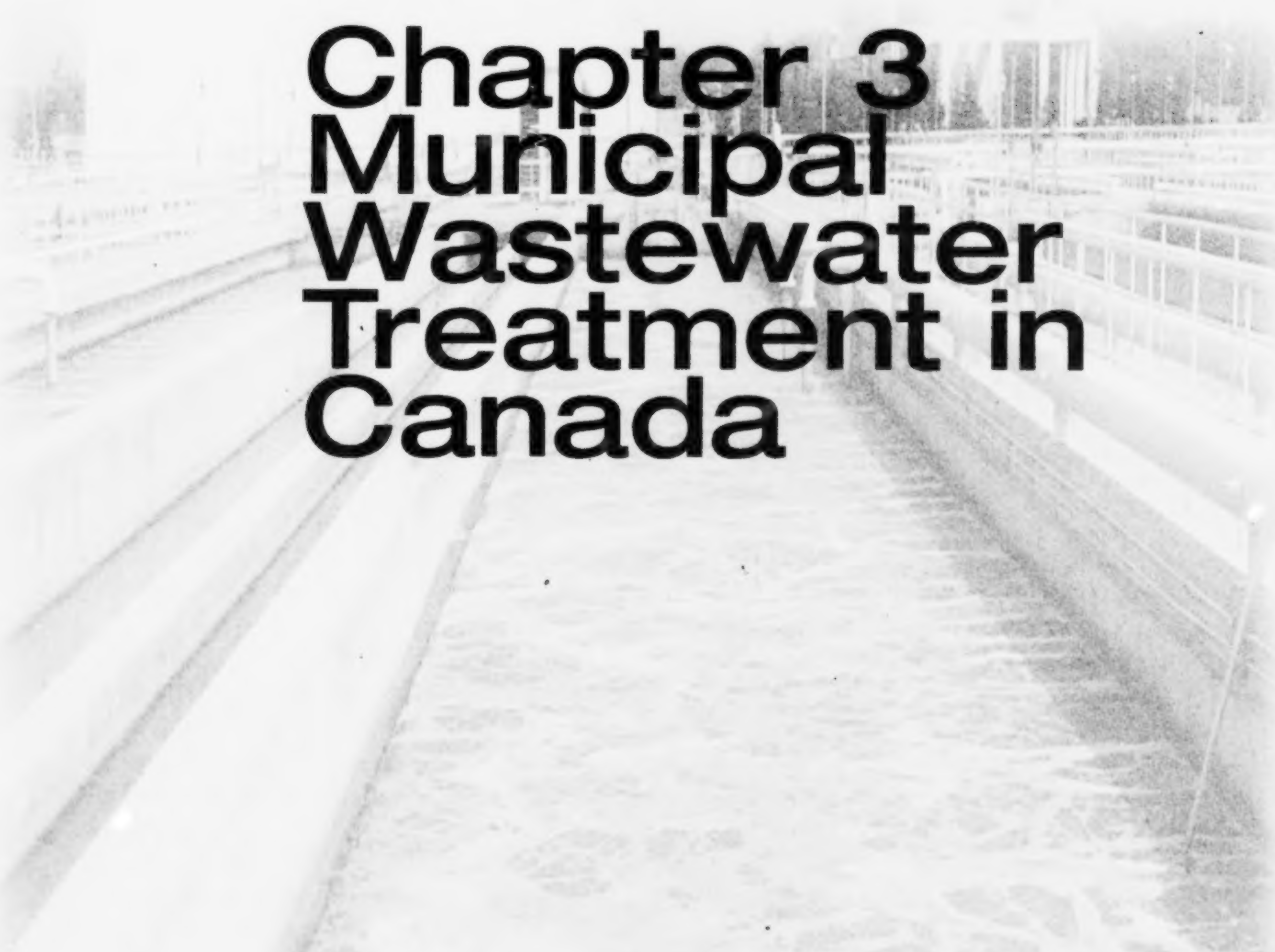
### 2.3.4 Public Health

Nitrate and nitrite represent a major public health concern in relation to methemoglobinemia and gastric cancer. Methemoglobinemia originates from the reduction of nitrate to nitrite in the stomach and saliva of infants. Ferrous iron in the hemoglobin molecules is oxidized with nitrite to ferric iron. Thus, the methemoglobin with iron in the ferric state cannot exchange oxygen, and suffocation may occur. Methemoglobinemia cases associated with water can occur at nitrate concentrations of greater than 45 mg/L (i.e., 10 mg $NO_3$-N/L) (USEPA, 1993). This level of nitrate is often exceeded in drinking water wells located in rural areas with septic systems for sewage treatment.

Nitrate is regarded as a potential factor in gastric cancer caused by the reduction of nitrate to nitrite, which can then react with amines and amides to form nitrosamine and nitrosamide compounds.

# Chapter 3

# Municipal Wastewater Treatment in Canada



# Chapter 3
# Municipal Wastewater Treatment in Canada

This chapter provides a characterization of municipal wastewater and associated treatment processes utilized in Canada. It begins with an overview of reported MWWTP influent and treated effluent characteristics. A summary is provided of the various types of wastewater treatment processes applied across Canada for service populations of greater than 500 persons. Process upgrade requirements for the enhanced control of total ammonia-nitrogen are briefly discussed.

## 3.1 Background

Municipal wastewater effluents contribute largely to the total volumetric loading of wastewater generated and discharged in Canada. Although more than 80% of the Canadian population is served by municipal wastewater treatment facilities, adverse impacts of effluent discharges on some lakes, rivers and coastal waters have been reported (Chambers *et al.*, 1997). Depending on the degree of treatment provided, municipal wastewater effluents can result in acute and chronic toxicity to organisms in receiving waters. In many cases, this is attributed to un-ionized ammonia and to total residual chlorine in cases where wastewater disinfection by chlorination is practised (OMOE, 1990, 1992; WTC and VPISU, 1994; Chambers *et al.*, 1997).

### 3.1.1 Sources of Municipal Wastewater

Municipal wastewater is derived from domestic, industrial, commercial and institutional sources. Infiltration and inflow can contribute to the volumetric loading to MWWTPs, especially in cases with combined sewer systems (i.e., where stormwater is combined with wastewater). The relative volumetric and contaminant mass loading contributions from these different sources are site specific and influence the untreated wastewater characteristics. Differences in characteristics depend on socioeconomic factors, the type of industry and associated effluents contributing to the MWWTP influent stream and the size, retention time and storage capacity within the wastewater collection system.

### 3.1.2 Characteristics of Municipal Wastewater

The reported average per capita daily wastewater flow rate from MWWTPs in Canada was approximately 0.3 $m^3$/capita in 1992 (Chambers *et al.*, 1997), although flow rates of about 0.5 $m^3$/capita per day are considered typical, with levels of up to 1.4 $m^3$/capita per day for some Canadian MWWTPs (Environment Canada, 2000a). Other reported information indicates that influent flow rates of 0.17 $m^3$/capita per day can be expected from residential sources (USEPA, 1992) and about 0.26 $m^3$/capita per day when infiltration and inflow are included (USEPA, 1993).

A 1986 survey of MWWTPs in the United States revealed a typical average influent flow rate of about 0.6 $m^3$/capita per day, which included typical domestic, industrial and commercial contributions as well as infiltration and inflow (USEPA, 1993).

Influent to MWWTPs consists of suspended solids, microorganisms and a wide variety of dissolved organic and inorganic chemical constituents. Those suspended and dissolved constituents that exert an oxygen demand can be measured as total 5-day biochemical oxygen demand ($BOD_5$) and chemical oxygen demand (COD). Contaminant generation rates include 0.08–0.11 kg/capita per day for both total suspended solids (TSS) and $BOD_5$ (USEPA, 1993) and 9–14 g/capita per day of total Kjeldahl nitrogen (TKN) (WEF, 1998). TSS, $BOD_5$, nutrients in the form of phosphorus, ammonium-nitrogen and total nitrogen (i.e., total ammonia-nitrogen, organic nitrogen, nitrate-nitrogen and nitrite-nitrogen), pathogenic bacteria, and plastics and floatables are typically the main targets for removal through wastewater treatment plants.

As defined in Metcalf & Eddy (1991), the typical composition of a medium-strength untreated municipal wastewater is considered to be:

- 220 mg TSS/L;
- 220 mg $BOD_5$/L;
- 500 mg COD/L;
- 40 mg TKN/L, consisting of 25 mg total ammonia-nitrogen/L and 15 mg organic nitrogen/L;
- 8 mg total phosphorus/L; and
- $10^7$–$10^9$ total coliform/L.

The total nitrogen concentration in raw MWWTP influent ranges from 15 to 85 mg/L, with 60% as total ammonia-nitrogen and the remainder essentially in organic form (Reed, 1985; WEF, 1998).

A comprehensive database generated during a pilot monitoring study at 37 Ontario MWWTPs in 1987 indicated an overall average influent strength that was somewhat lower than the aforementioned medium-strength characteristics. Based on data for the various plants, the calculated mean influent concentration included 125 mg TSS/L, 140 mg $BOD_5$/L, 290 mg COD/L, 25 mg TKN/L, 15 mg total ammonia-nitrogen/L and 5 mg total phosphorus/L (OMOE, 1988). However, the concentration of influent between the different plants varied widely, with 22% of the sampled plants having average $BOD_5$ concentrations greater than the 220 mg/L value defined above for a typical medium-strength wastewater.

Results of a recent short-term sampling and monitoring survey of 28 MWWTPs across Canada in 1998–99 revealed that 39% of the selected plants had influent $BOD_5$ concentrations of greater than 220 mg/L (Environment Canada, 2000a, 2000b). The calculated average influent concentrations were 190 mg TSS/L, 205 mg $BOD_5$/L, 36 mg TKN/L and 20 mg total ammonia-nitrogen/L. The range of concentration measurements over all the plants sampled was wide, including 30–420 mg TSS/L, 40–625 mg $BOD_5$/L and 9–75 mg TKN/L.

Influent total ammonia-nitrogen concentrations can sometimes be influenced by the contaminant loading associated with liquid recycle streams from the solids processing stages of mechanical MWWTPs (e.g., supernatants and filtrates from digestion and dewatering processes, respectively). These streams can contain elevated levels of total ammonia-nitrogen if anaerobic digestion of sludge is practised. The recycle streams are typically returned upstream of the primary treatment stage of the liquid train. The associated loading of nitrogen can increase the raw wastewater nitrogen concentration by up to 20–30% (WEF, 1998) and should be given consideration in the design of ammonia removal processes. Another consideration is that for small to medium-sized plants, the return stream flows and loadings may be variable owing to the discontinuous nature of the solids processing operations (USEPA, 1993).

### 3.1.3 Treated Municipal Wastewater

There were more than 2800 MWWTPs in Canada in 1994. Of the Canadian population that was connected to wastewater treatment facilities at that time, about 25% were served by primary treatment plants, and the remaining 75% were served by biological process-based secondary or tertiary treatment systems (Chambers *et al.*, 1997).

Primary treatment plants are utilized for the removal of suspended solids from wastewater, with modest corresponding reductions in BOD. Secondary treatment enhances the removal of BOD, including the biological degradation of soluble organic contaminants and, depending on the applied process conditions, total ammonia-nitrogen through nitrification and/or assimilation. For example, extended aeration activated sludge plants can typically achieve virtually complete removal of total ammonia-nitrogen (i.e., <1 mg/L). Tertiary treatment operations enhance control over discharge concentrations of other specific chemical contaminants, such as total phosphorus. The degree of treatment employed generally depends on site-specific receiving water characteristics and the regulatory requirements governing effluent discharges.

The chemical composition of MWWTP effluent discharges can vary greatly from plant to plant, depending on the level and type of treatment utilized, the relative fractions of domestic and industrial wastewater contributions to the influent, the type of industries, surface area served, etc. Temporal variability of effluent characteristics can be experienced at a given plant. This can be associated with daily and weekly variations in influent composition as well as seasonal variations in industrial and domestic contributions or in treatment plant process conditions.

MWWTP effluent discharge concentrations of TKN and total ammonia-nitrogen are greatly affected by plant loading or the level of wastewater treatment employed and whether biological nitrification is being achieved. For instance, during the monitoring study of the 37 Ontario MWWTPs in 1987, 47% of the 30 secondary or tertiary treatment plants discharged total ammonia-nitrogen concentrations of less than 5 mg/L. The average TKN and total ammonia-nitrogen concentrations in the secondary/tertiary-treated effluent samples were 8 and 4 mg/L, respectively. Furthermore, of the 25 secondary/tertiary treatment plants that were sampled as part of the nationwide survey conducted for Environment Canada in 1998–99, 44% were found to discharge total ammonia-nitrogen concentrations of below 5 mg/L. The short-term daily composite sampling campaigns were conducted in autumn to early winter (Environment Canada, 2000a, 2000b).

Effluent quality from lagoon-based secondary treatment processes is susceptible to significant seasonal effects due to large surface areas and heat losses. Although effluent total ammonia-nitrogen concentrations of less than 5 mg/L are possible from continuous-discharge facultative and aerated lagoon systems during summer and autumn operating periods in Canada due to elevated temperatures and biological activity, considerably higher residual concentrations are typically experienced during cold winter periods and in the springtime (OMOE and Environment Canada, 1993; NovaTec Consultants Inc., 1996). Effluent discharge characteristics from fill-and-draw-type conventional facultative lagoon systems in Canada typically involve low total ammonia-nitrogen concentrations (i.e., <5 mg/L) from annual autumn discharge plants. However, seasonal facultative lagoons with twice-per-year discharge periods typically release considerably higher levels of total ammonia-nitrogen (i.e., 8–25 mg/L) during the springtime due to the effects of ice cover and reduced surface aeration during the wintertime, among other effects (Alberta Environment, 1987; OMOE and Environment Canada, 1993).

## 3.2 Municipal Wastewater Treatment Plants

### *3.2.1 Treatment Plant Installations in Canada*

Current information on Canadian MWWTP installations designed for populations of greater than 500 persons was collected through contacts with government representatives from the various provinces and territories and from reported data (Ministry of Supply and Services Canada, 1987; BCMOE, 1991; Alberta Environment, 1996) and databases available for some of the provinces. This population limit is based on the U.S. EPA's definition (USEPA, 1992) of very low flow rate treatment plant applications being those with processing rates of less than 50 000 U.S. gallons/day (i.e., <200 $m^3/d$), which approximately corresponds to a population of 500 persons.

The information available on the number of treatment facilities in the province of Manitoba was irrespective of the population served, since there were no plant-specific population data available. The number of MWWTPs in Manitoba has been included in the summary of the total number of plants in Canada but not in the population-based diagrams to be subsequently illustrated. Information on treatment plants installed at resorts and other recreational facilities in Canada was not included in this characterization.

The range of municipal wastewater treatment capacity in Canada is broad. The majority of Canadian MWWTPs are categorized as small, having design wastewater flow rates of less than 5000 $m^3/d$ (i.e., <5 MLD or <1 MIGD); however, there are also numerous medium- (i.e., 5–50 MLD or 1–11 MIGD) and large-capacity (i.e., >50 MLD or >11 MIGD) installations. A total population of 20 157 300 persons is currently serviced by the MWWTPs in communities having greater than 500 people.

MWWTPs in the small-capacity category generally consist of facultative lagoons or aerated lagoon-based secondary treatment. In Ontario, 90% of lagoon-based treatment processes have reported design capacities of <3.3 MLD (OMOE and Environment Canada, 1993). The relatively low capital and operating costs, low degree of complexity and O&M requirements and the large and inexpensive land areas that are generally available in the vicinity of small communities make lagoon systems an attractive option. Extended aeration activated sludge plants are also frequently employed for small applications and offer a higher degree of process control and consistently high effluent quality.

Conventional facultative lagoons represent the most commonly used form of municipal wastewater treatment in Canada, making up nearly 80% of all MWWTPs in the western provinces and the territories. Conventional facultative lagoons typically comprise multiple treatment cells arranged in series for solids and BOD removal. They can be designed for continuous-discharge operation or can include a facultative storage pond to permit controlled seasonal discharge during spring and/or autumn. The requirement for storage capacity usually depends on the winter climatic conditions and the receiving water characteristics and constraints.

Aerated lagoons are in widespread use across Canada. They are smaller in size and deeper than conventional facultative lagoons and consist of one or more aerated cells arranged in series, and they include a final quiescent zone within the last cell to permit solids settling. Oxygen is supplied by mechanical means using either mechanical surface aerators or diffused aeration. Aerated lagoons can be either partially mixed or completely mixed. They are operated as continuous-discharge systems and offer a somewhat greater degree of process control than do facultative lagoons. Most aerated lagoons are located in the province of Québec, making up 80% of the treatment plants there. Aerated facultative lagoons are another common process configuration. They include a mechanically aerated cell followed by a larger facultative cell, also typically operated in continuous-discharge mode.

The majority of installations with medium to large treatment capacity are mechanical secondary treatment plants utilizing the activated sludge process. Primary treatment installations make up a number of large-scale MWWTP applications. An example is the chemically assisted primary treatment facility utilized by the Montréal Urban Community, which serves a population of 1.8 million, with a corresponding average flow rate of 2500 MLD (i.e., 550 MIGD).

The survey of current information on Canadian MWWTP installations is summarized in Table 3.1. MWWTPs are classified as either primary or secondary treatment processes, according to the nature and level of the contaminants removed (Section 3.1.3). Primary treatment can involve medium and fine screening, grit removal, primary clarification and comminution, with or without chemical pre-precipitation. The secondary treatment category is divided into conventional facultative lagoons, low-rate mechanical aerated lagoon systems and other mechanical plants utilizing higher-rate biological processes, which are individually listed. Table 3.1 summarizes the number and type of these treatment processes utilized in Canada according to the above-mentioned classifications and categories.

The conventional lagoons category represents those facultative systems that do not employ mechanical equipment for aeration but rather rely on surface aeration and photosynthetic activity for oxygen supply and may include up-front anaerobic ponds. The lagoon-based mechanical systems category consists of either aerated lagoons with all cells aerated or aerated facultative lagoon systems. The higher-rate mechanical secondary treatment systems include

### Table 3.1 Canadian Municipal Wastewater Treatment Plant Overview

| Municipal Wastewater Treatment Plants[1] | | | Number |
|---|---|---|---|
| Primary treatment | | | 90 |
| Secondary treatment | Conventional facultative lagoons | | 762 |
| | Low-rate mechanical systems (i.e., aerated lagoons and aerated facultative lagoons) | | 593 |
| | Higher-rate mechanical systems | Conventional activated sludge | 212 |
| | | High-rate activated sludge | 11 |
| | | Extended aeration activated sludge | 157 |
| | | Contact stabilization activated sludge | 22 |
| | | Oxidation ditch | 63 |
| | | Sequencing batch reactor (SBR) | 23 |
| | | Membrane bioreactor (MBR) | 4 |
| | | Rotating biological contactor (RBC) | 34 |
| | | Trickling filter | 29 |
| | | Biological aerated filter (BAF)[2] | 9 |
| Total MWWTPs | | | 2009 |

[1] Summary of all treatment plants serving populations of greater than 500 persons. Note: since the database available for Manitoba did not have a cut-off for the population served, all treatment plants in that province were included in the summary (i.e., including those serving fewer than 500 persons).

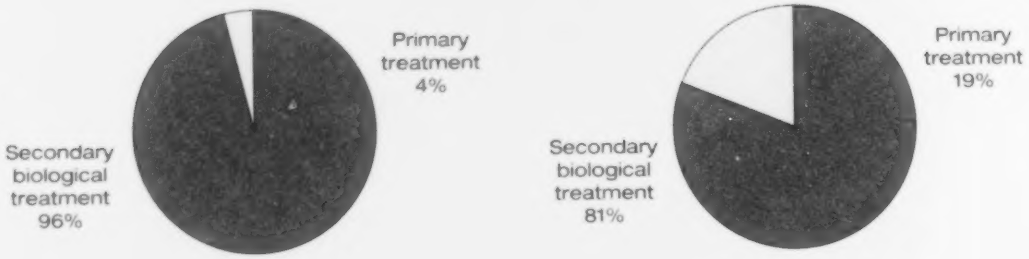
[2] BAF technology is also referred to as aerobic submerged fixed-bed reactor.

various suspended growth and attached growth aerobic processes, as listed in Table 3.1.

The relative prevalence of MWWTPs, shown according to the level and type of treatment processes, is presented in Figures 3.1, 3.2 and 3.3. The total number of treatment plants in a given category serves as the basis for the pie charts in Figures 3.1a, 3.2a and 3.3a. The relative distribution of treatment level and process type on a population served basis is shown in Figures 3.1b, 3.2b and 3.3b and reflects the importance of the type of treatment plants without consideration of the number of plants. The population-based figures used for these diagrams were calculated without considering the treatment plants in the province of Manitoba, as the corresponding population data were not available.

Figure 3.1 presents the distribution of primary and secondary treatment plants in Canada. It shows that although primary treatment makes up only 4% of the total number of MWWTPs, this level of treatment services nearly 20% of the Canadian population. Most of the primary MWWTPs are found in the provinces of Québec and British Columbia. About 80% of

**Figure 3.1 Primary and Secondary Treatment Plants**



a) Distribution based on the total number of primary and secondary treatment plants

b) Distribution based on the population served by the primary and secondary treatment plants (excluding Manitoba)

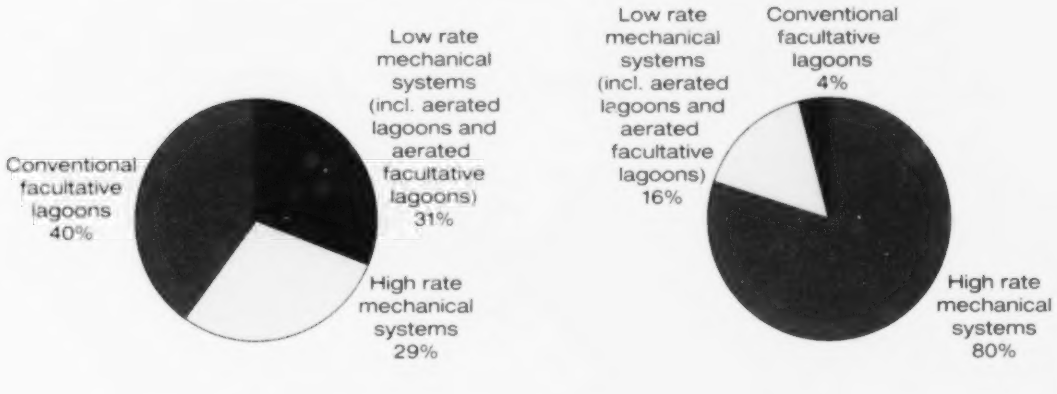
the Canadian population served by primary treatment plants is located in these two provinces.

In terms of high-rate secondary treatment, various types of activated sludge and attached-growth processes are found throughout the country, with some types of treatment more prevalent in certain provinces. For instance, SBR processes are found mainly in Québec, Ontario and Manitoba, while trickling filters are especially applied in British Columbia, and BAF installations are found mainly in Québec.

Figure 3.2 summarizes secondary treatment installations in Canada and is divided into low-rate and high-rate mechanical systems as well as conventional facultative lagoons. Lagoon-based systems in general, and conventional facultative lagoons in particular, are the most commonly used type of secondary treatment for municipal wastewater in Canada, as shown in Table 3.1. Conventional lagoons generally serve communities with fewer than 2000 people but can be applied to larger municipalities, such as Brooks, Alberta, with a population of 9500. Although conventional facultative lagoons represent 40% of all secondary treatment plants in Canada (Figure 3.2a), the relatively small population base served by such plants corresponds to a disproportionately small percentage of 4% in terms of the total population served by conventional lagoons in comparison with mechanical secondary treatment plants (Figure 3.2b).

A further breakdown of secondary treatment based only on lagoon processes (i.e., conventional facultative lagoons versus low-rate mechanical aerated lagoons) is presented in Figure 3.3. The two types of lagoon processes together make up about 70% of the total number of secondary treatment plants in Canada, based on data in Table 3.1. Although lagoon-based mechanical systems are typically applied in small rural and semirural communities for relatively small wastewater flows, there are examples of medium- and large-scale systems, such as the 75 MLD

**Figure 3.2 Secondary Treatment — Conventional Facultative Lagoons and Low-Rate and High-Rate Mechanical Systems**



a) Distribution based on the total number of conventional facultative lagoons and low-rate and high-rate mechanical treatment plants

b) Distribution based on the population served by conventional facultative lagoons and low-rate and high-rate mechanical treatment plants (excluding Manitoba)

(or 16.5 MIGD) aerated lagoon in Regina, Saskatchewan. This is reflected in a comparison of Figures 3.3a and 3.3b.

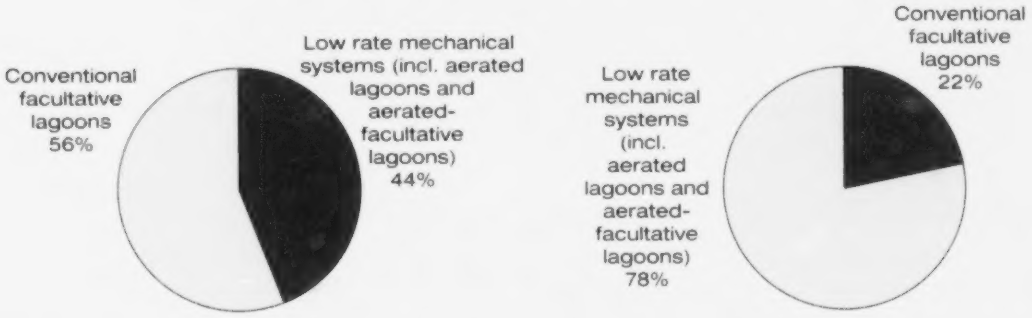
The distinction between secondary and tertiary levels of treatment as well as the site-specific requirement for ammonia removal were not clearly defined or provided in the information obtained as part of the survey on Canadian MWWTPs. There are certain cases where ammonia limits exist based on water quality objectives for downstream use, mixing zone percentage (i.e., percentage of the stream flow used for dilution) and allocation factors (i.e., percent allowable dilution allocated to individual dischargers).

Chemical phosphorus removal is practised on a seasonal or year-round basis in several primary and secondary treatment plants in the provinces of Québec, Ontario and British Columbia. Biological treatment plants currently utilizing or having the potential for biological nutrient removal are found in the provinces of Alberta (two plants), British Columbia (six plants, with three using the Bardenpho process), Ontario (three plants) and Québec (two plants).

### 3.2.2 Treatment Process Capabilities and Considerations

The control or removal of ammonia or total nitrogen is currently not a general regulatory requirement for MWWTPs in Canada. Plants utilizing solely preliminary, primary or chemically assisted primary treatment do not have the

**Figure 3.3 Lagoon-Based Secondary Treatment — Conventional Facultative Lagoons and Low-Rate Mechanical Systems**



a) Distribution based on the total number of conventional facultative lagoons and low-rate mechanical treatment plants

b) Distribution based on the population served by conventional facultative lagoons and low-rate mechanical treatment plants (excluding Manitoba)

capability of removing ammonia, although some reduction of TKN is achieved with the removal of suspended solids. Furthermore, the majority of secondary treatment plants are designed mainly for enhanced BOD removal along with the control of TSS discharges. However, conventional secondary treatment plants can achieve some degree of total ammonia-nitrogen removal through assimilation, volatilization and/or nitrification, depending on process design and operating conditions.

Conventional facultative lagoons that are designed with the provision for effluent storage and a controlled annual autumn discharge are capable of achieving low residual levels of total ammonia-nitrogen (i.e., <5 mg/L) in the effluent stream (OMOE and Environment Canada, 1993). Conversely, although continuous-discharge lagoon systems can discharge low concentrations of ammonia during summer and autumn periods, elevated total ammonia-nitrogen discharge concentrations are typically experienced at other times of the year due to significant seasonal effects (OMOE and Environment Canada, 1993; NovaTec Consultants Inc., 1996). In summary, it is not considered to be feasible to design continuous-discharge or seasonal-discharge (i.e., twice per year) lagoon treatment systems for cold-climate applications that can reliably maintain effluent total ammonia-nitrogen concentrations of less than 5 mg/L.

Upgrade measures would be required to enhance the quality of lagoon effluent to achieve ammonia-nitrogen discharge concentrations of less than 5 mg/L on a consistent basis, if necessary. One approach that has been used for upgrading the quality of lagoon effluent is intermittent sand filtration technology for seasonal polishing of lagoon discharges (OMOE and Environment Canada, 1993; McMaster University and WTI, 1996). However, this technology is applicable only for the enhancement of effluent from seasonal-discharge lagoons.

The large number of conventional facultative lagoons and mechanical aerated lagoon treatment plants reveals the potential widespread requirement for process upgrading to achieve effluent total ammonia-nitrogen removal to below a 5 mg/L maximum monthly target discharge concentration on a consistent basis, if necessary. This would apply mainly if consideration were given only to total ammonia concentration in the end-of-pipe effluent. Treatment requirements may be less stringent if ammonia discharge regulations were to give consideration to receiving water conditions such as low wintertime temperatures, which would result in a reduced fraction of the toxic un-ionized form of ammonia and corresponding increased allowance in total ammonia-nitrogen discharge concentration.

Higher-rate mechanical secondary treatment plants that are designed and operated at relatively short solids retention times (SRT) and hydraulic retention times (HRT), elevated food-to-microorganisms ratio (F:M) and limited DO concentrations (i.e., <2 mg/L) can achieve minimal, if any, nitrification and only modest removals of total ammonia-nitrogen. Conversely, those suspended-growth and attached-growth mechanical secondary treatment plants that are designed to operate at lower loading rates can achieve partial or complete nitrification and, therefore, increased levels of ammonia removal. This is often dependent on seasonal influences such as process temperature. Total ammonia-nitrogen removal is not significantly influenced by chemical addition and/or tertiary filtration, which is practised at a number of MWWTPs in Canada for the enhanced control of soluble and total phosphorus discharges.

Extended aeration activated sludge plants are generally capable of supporting a nitrifying population, owing to elevated SRT, HRT and DO levels, and can potentially achieve complete year-round nitrification (OMOE, 1984; OMOE and Environment Canada, 1993). Upgrading of the secondary clarification stage may be required for some existing extended aeration plants to ensure effective control of SRT to levels required for year-round nitrification (OMOE, 1990).

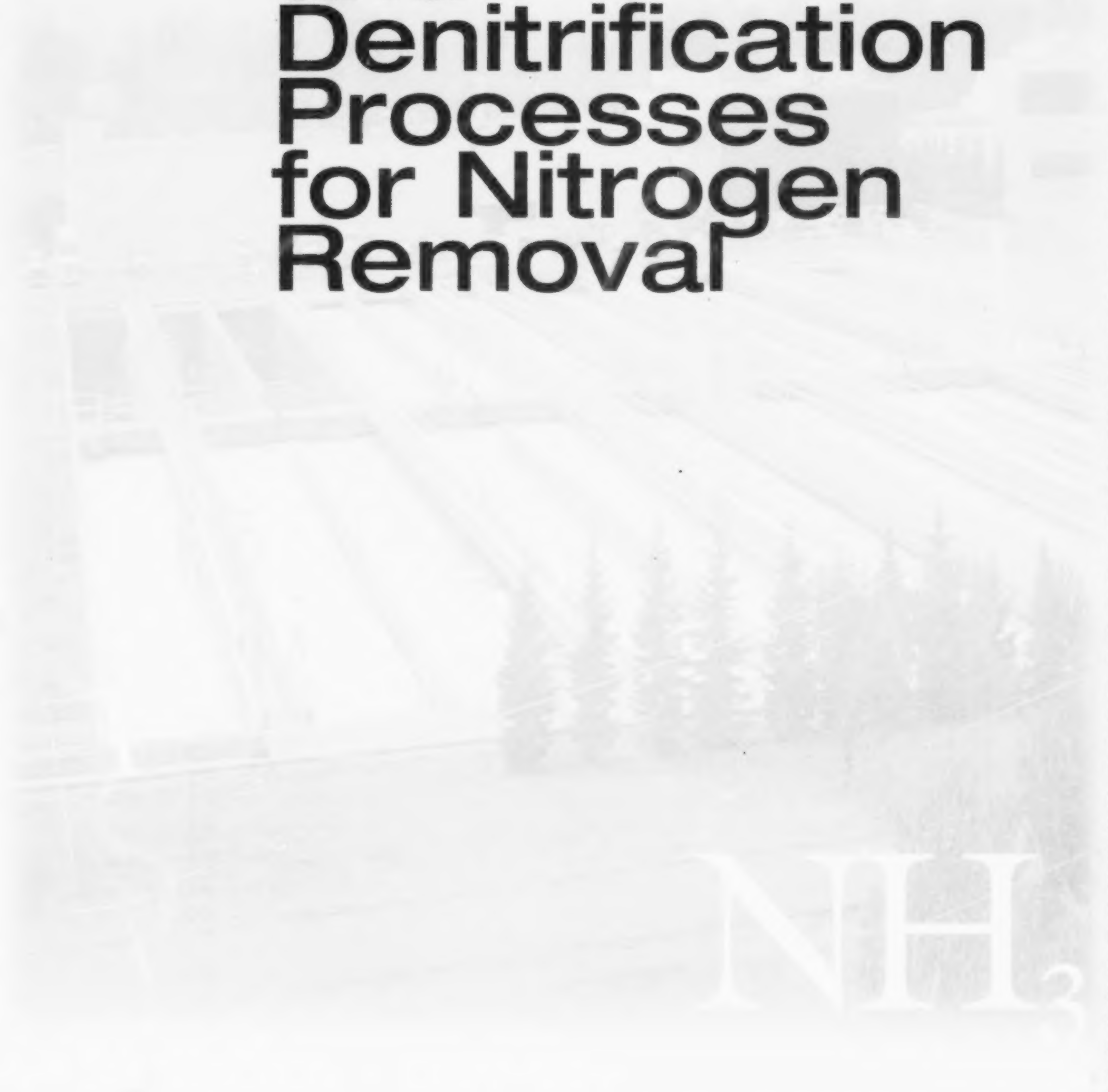
As described in the previous section, a number of biological nutrient removal plants have recently been installed in Canada, as either process retrofits or greenfield installations. Biological nutrient removal processes are inherently designed to achieve complete and consistent nitrification.

Since effluent discharges from most MWWTPs in Canada represent significant sources of total ammonia-nitrogen release to the receiving environment, upgrade measures may be required for enhanced treatment process performance. This will depend on eventual regulatory limits on effluent ammonia discharges and on site-specific conditions and receiving water characteristics.

Upgrade measures can include process optimization, plant retrofits or expansions, the installation of add-on processes or the replacement of existing systems with new processes to achieve year-round complete nitrification. In particular, systems that are limited to preliminary, primary or chemically assisted primary treatment will require a relatively high degree of upgrading. Similarly, relatively large continuous-discharge aerated lagoon processes may require significant upgrading or replacement, since the installation of downstream storage ponds would be infeasible due to large land area requirements. A number of MWWTPs in Canada have already been upgraded for enhanced control of total ammonia-nitrogen and/or total nitrogen discharges or are currently in the planning stages for installing enhanced nutrient removal capabilities (e.g., the MWWTPs in the City of Winnipeg). Chapter 6 provides an overview of potential upgrade measures applicable for various treatment scenarios and the different types of existing treatment processes.

# Chapter 4

# Fundamentals of Nitrification and Denitrification Processes for Nitrogen Removal



# Chapter 4
# Fundamentals of Nitrification and Denitrification Processes for Nitrogen Removal

This chapter reviews fundamental aspects of biological processes involved in the removal of nitrogen from wastewater. This information is referenced later in the report, particularly in Chapter 5, which describes specific nitrification and denitrification process technologies, and in Chapter 6, which covers important factors and considerations in the selection of process alternatives for specific MWWTP applications.

## 4.1 Nitrogen Chemistry

Nitrogen is an essential nutrient for biomass growth, as it is found in proteins and nucleic acids. Considering a typical activated sludge biomass composition of $C_5H_7NO_2$, a nitrogen content of 0.12 g/g volatile suspended solids (VSS) (i.e., 12%) is typically calculated.

Nitrogen can have oxidation states of any value between −3 and +5, with the most common forms presented in Figure 4.1. The major forms of nitrogen found in the input (influent) and outputs (effluent, sludge, gas) of a biological process are illustrated in Figure 4.2.

## 4.2 Nitrogen Cycle in Wastewater Treatment

Significant nitrogen cycle processes are illustrated in Figure 4.1 and are discussed below.

### 4.2.1 Hydrolysis

The degradation of organic matter into basic components (e.g., ammonium) is referred to as hydrolysis. Other terms, such as "organic matter lysis," "ammonification" or "biomass endogenous respiration," are globally referred to as hydrolysis. In a biological wastewater treatment process, influent organic matter is hydrolysed so that the biomass can "burn" organic matter for the production of energy, which is used for cellular maintenance and biomass synthesis (which also uses organic matter as cellular building blocks).

### 4.2.2 Synthesis

For biomass synthesis, nitrogen is assimilated as ammonium by most living organisms, except for green plants and algae, which use nitrate as a nitrogen source.

### 4.2.3 Nitrification

Nitrification is considered to be a two-step process consisting of total ammonia oxidation or nitritation (ammonium to nitrite) and of nitrite oxidation or nitratation (nitrite to nitrate). Nitritation is considered to be achieved by bacteria such as *Nitrosomonas* and nitratation by *Nitrobacter*, both being autotrophs (i.e., using inorganic carbon, carbon dioxide, as a carbon source for cell synthesis). Bacterial growth requires that microbial cells obtain energy to produce cellular material. Nitrifying bacteria will grow only in environments in which DO is present; however, the absence of DO for prolonged periods is not lethal (WEF, 1998).

#### 4.2.3.1 Stoichiometry

For ammonium oxidation to nitrate by *Nitrosomonas* and *Nitrobacter*, the overall energy-producing reaction is:

$$NH_4^+ + 2\,O_2 \rightarrow NO_3^- + 2\,H^+ + H_2O \qquad (4.1)$$

**Figure 4.1 Nitrogen Pathways of Significance in Wastewater Treatment**

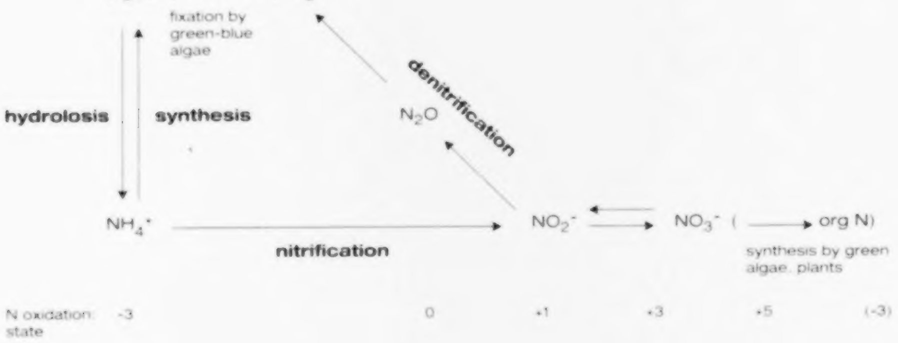
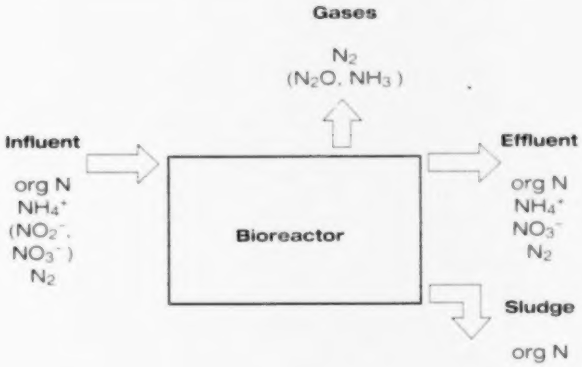

org N
N2
fixation by green-blue algae
hydrolosis
synthesis
denitrification
N2O
NH4+
nitrification
NO2-
NO3-
org N)
synthesis by green algae, plants
N oxidation state
-3
0
+1
+3
+5
(-3)

**Figure 4.2 Nitrogen Forms of Significance in Wastewater Treatment**


Gases
N2
(N2O, NH3)
Influent
org N
NH4+
(NO2-,
NO3-)
N2
Bioreactor
Effluent
org N
NH4+
NO3-
N2
Sludge
org N

An equation for cell growth can be written based on the above reaction. Accounting for cell yields, an overall energy production and synthesis equation for ammonium oxidation into nitrate can be presented as:

$$1.00\ NH_4^+ + 1.89\ O_2 + 0.0805\ CO_2 \rightarrow 0.0161\ C_5H_7NO_2 + 0.984\ NO_3^- + 1.98\ H^+ + 0.952\ H_2O \qquad (4.2)$$

From this equation, a number of stoichiometric relationships can be derived for oxygen and alkalinity consumption and biomass growth, which are summarized in Table 4.1.

#### 4.2.3.2 Kinetics

The rate (i.e., kinetics) of nitrification (i.e., involving nitrifier growth and ammonium oxidation) can be affected by a number of factors, which will impact directly on the design and operational characteristics of nitrification processes.

The maximum specific growth rate of nitrifiers (i.e., about 0.3–1.2/day) is considered to be 10–20 times slower than that of heterotrophic bacteria that are responsible for the stabilization of organic matter. For this slow growth rate, a sufficiently long SRT must be provided to obtain stable nitrification.

Ammonia and DO must be available for nitrification to occur, as indicated in equation 4.2, and a limiting concentration of either would reduce the rate of nitrification. All other factors being optimal, an actual ammonium-nitrogen concentration of 0.6–3.6 mg/L would result in the growth rate of nitrifiers being one-half of its maximum value. For DO, a concentration of 0.4–2.0 mg/L would reduce by one-half the maximum rate of nitrification. For efficient nitrification, it is generally recommended that the DO concentration be maintained at a value of at least 2 mg/L.

In single-sludge systems aiming at total nitrogen and phosphorus removal, the biomass is cyclically exposed to non-aerated and aerated zones. Studies have shown that the absence of DO for up to 5 hours had little effect on the rate of nitrification. Transient organic shock loads, however, may affect nitrification due to the competition for oxygen to achieve nitrification by nitrifiers and the stabilization of organic matter by heterotrophic microorganisms. In such cases, nitrite may accumulate, and it is advisable to increase the aeration rate.

The optimal pH for nitrification is in the range of 7.0–8.5, but nitrification can take place in the range of 6.0–10.0 standard pH units. As free acid ($H^+$) is produced (equation 4.2), nitrification will tend to reduce the alkalinity and eventually the pH of the suspension. Should insufficient alkalinity be available to buffer the acidity production, the rate of nitrification would be reduced as the pH decreases. If required, sources of alkalinity may be added (Table 4.1) to ensure an effluent calcium carbonate concentration of at least 50–100 mg/L. Aeration also contributes to pH increases by stripping carbon dioxide from solution. The effect of pH on nitrification appears to be more an inhibition phenomenon than a toxic shock effect, as indicated by a rapid recovery in the nitrification rate when pH conditions are brought back into the optimal range. Acclimation to suboptimal pH conditions can attenuate the inhibition of pH but will not result in nitrification rates as high as in the optimal pH range.

Temperature significantly affects the growth rate of nitrifiers. Nitrification occurs over a range of about 4–45°C, with an optimum temperature range of about 30–35°C. In the range of 5–30°C,

**Table 4.1 Stoichiometry of Reactions**

| Processes | Stoichiometry of Reactions |
|---|---|
| Nitrification | 4.6 mg $O_2$ utilized/mg total ammonia-nitrogen oxidized |
| | 7.1 mg $CaCO_3$ alkalinity consumed/mg total ammonia-nitrogen oxidized |
| | 0.10–0.15 mg VSS (nitrifiers) formed/mg total ammonia-nitrogen oxidized |
| Alkalinity sources | 1.8 mg $CaCO_3$ alkalinity added/mg CaO (quicklime) added |
| | 1.4 mg $CaCO_3$ alkalinity added/mg $Ca(OH)_2$ (slaked lime) added |
| | 1.2 mg $CaCO_3$ alkalinity added/mg NaOH (caustic) added |
| | 0.9 mg $CaCO_3$ alkalinity added/mg $Na2CO_3$ (soda) added |
| Biological denitrification | 3.6 mg $CaCO_3$ alkalinity recovered/mg nitrate-nitrogen reduced |
| | similar or slightly lower biomass yield as for aerobic heterotrophic stabilization of organic matter |
| Chlorination (breakpoint) | 7.6 mg $C1_2$/mg ammonium-nitrogen oxidized (10–30% more in practice due to competing reactions) |
| | 1.4 mg $CaCO_3$ alkalinity loss/mg $C1_2$ added |
| | 5.1 mg $C1_2$/mg nitrite-nitrogen oxidized into nitrate |

the reaction rate will double with each 7°C increment in nitrification process temperature.

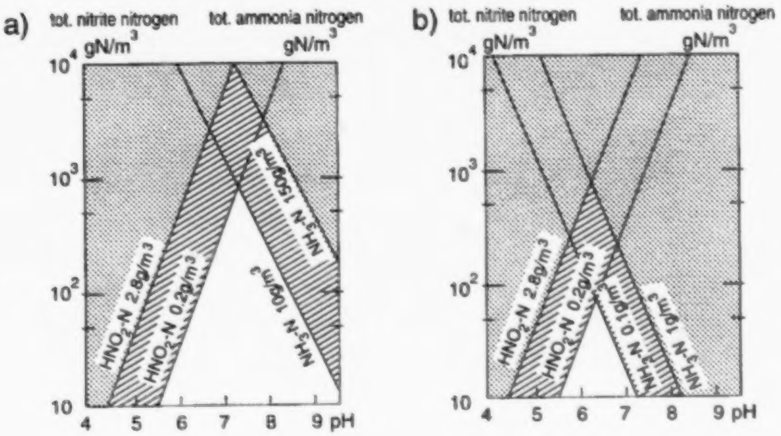
#### 4.2.3.2.1 Inhibition Effects

Inhibition of nitrifiers may occur due to the presence of certain inorganic (e.g., un-ionized ammonia, nitrous acid [$HNO_2$], metals) and organic chemicals. Inhibition by chemicals would normally result not in a complete loss of the ability of the process biomass to nitrify, but in a gradual washout of the nitrifiers. However, depending on the specific substance, its concentration, the duration of exposure and other environmental conditions, effects on the sensitive nitrifying population could be fatal (WEF, 1998).

Inhibitory concentrations of specific metals and inorganic and organic substances are listed in Appendix B. The presence of a number of inhibitory substances may have a synergistic effect, which can result in toxic effects appearing at concentrations that are lower than the threshold levels for the respective substances. Care should be taken, however, when interpreting reported concentrations of inhibitory substances, since acclimation can occur over time of exposure and reduce or effectively remove the inhibitory effect from a biological system (USEPA, 1993). The reported data can be used as references for the relative effect of specific substances.

Inhibition of *Nitrosomonas* (responsible for nitritation) by free ammonia can occur at concentrations of 5–150 mg nitrogen/L, while inhibition of *Nitrobacter* (responsible for nitratation) is possible at concentrations of 0.1–1 mg nitrogen/L (USEPA, 1993; WEF, 1998). Nitrous acid is toxic to both *Nitrosomonas* and *Nitrobacter* at a concentration of 0.2–2.8 mg nitrogen/L. Figure 4.3 allows for the determination of which reactions are expected to be inhibited, based on pH levels and the concentration of total ammonia and total nitrite.

**Figure 4.3 Inhibition of *Nitrosomonas* (Nitritation; a) and of *Nitrobacter* (Nitratation; b) by Free Ammonia ($NH_3$) and Nitrous Acid ($HNO_2$) (from Henze *et al.*, 1995)**



The light zones are those without inhibition, the hatched zone are those with partial inhibition and the shaded zones are those with inhibition.

### 4.2.4 Denitrification

Denitrification results in the biological transformation of nitrate into nitrogen gas via nitrite and nitrous oxide ($N_2O$). This reaction also results in the stabilization of organic matter, just as would happen with oxygen. In fact, most denitrifying bacteria can use either nitrate or oxygen, preferring the latter when available, as it provides a greater energy gain for a given amount of organic matter stabilized.

#### 4.2.4.1 Stoichiometry

Theoretically, 1.0 g of nitrate-nitrogen is equivalent to 2.86 g of oxygen for the stabilization of organic matter. Thus, in comparison with the 4.57 g of oxygen required to nitrify 1 g of ammonium-nitrogen, 63% (i.e., 2.86/4.57) of the oxygen is credited back by denitrification.

The main sources of carbon used for denitrification are raw wastewater, fermented raw wastewater, fermented primary sludge, none (i.e., endogenous decay is relied on to provide carbon for denitrification), industrial wastewaters (e.g., brewery wastes, molasses), acetic acid and methanol ($CH_3OH$).

Using methanol as a supplemental source of organic substrate, the following equation has been proposed for denitrification with nitrate:

$$NO_3^- + 1.08\ CH_3OH + 0.24\ H_2CO_3 \rightarrow$$
$$0.056\ C_5H_7NO_2 + 0.47\ N_2 + 1.68\ H_2O + HCO_3^- \quad (4.3)$$

Thus, about 2.5 g of methanol would be required for the conversion of 1 g of nitrate-nitrogen. In practice, 20–100% more would need to be added, as any oxygen or nitrite present would require extra methanol.

Alkalinity is recovered by denitrification (3.57 mg calcium carbonate alkalinity/mg nitrate-nitrogen denitrified) to a level slightly less than 50% (i.e., 3.57/7.6) of that consumed by nitrification.

Stoichiometric relationships can be derived for alkalinity consumption and biomass growth, which are summarized in Table 4.1.

#### 4.2.4.2 Kinetics

The maximum specific growth rate of denitrifying bacteria (i.e., about 5 g VSS/g COD stabilized per day) is about 20% less than that of heterotrophic aerobic microorganisms that stabilize organic matter.

The actual nitrate concentration will affect the rate of denitrification, with a value of 0.5 mg nitrate-nitrogen/L, giving a rate of one-half of the maximum. Similarly, a source of organic matter is required, with a concentration of about 20 mg/L of soluble and readily biodegradable COD resulting in one-half of the maximum denitrification rate.

Conversely, the presence of oxygen would inhibit denitrification, as oxygen is a preferred compound for the oxidation of organic matter. It is considered that denitrification is completely halted at a DO concentration of greater than 1.0 mg/L, while it is at 50% of its maximum at 0.5 mg/L (some authors use 0.1 or 0.2 mg/L) and is maximal at 0.0 mg/L.

Incomplete denitrification may result in nitrous oxide formation. This compound is a gas that contributes to the greenhouse effect, 270 times more than carbon dioxide. Nitrous oxide may be produced in aerobic processes in which the organic loading is high enough to result in a residual oxygen concentration that is less than 0.5 mg/L. This condition allows for both slow nitrification and partial denitrification to take place concurrently.

The optimal pH for denitrification is in the range of 6.5–7.5.

The effect of temperature on denitrification is similar to effects on heterotrophic aerobic bacteria (i.e., doubling of the reaction rate for every 10°C increment in process temperature).

Inhibitors have a comparable effect on denitrifiers as on heterotrophic aerobic bacteria.

### 4.2.5 Other Processes

Other processes involving nitrogen conversion that are sometimes relevant to wastewater treatment are presented below.

#### 4.2.5.1 Synthesis by Plants and Algae Using Nitrate

Green plants and algae use nitrate as opposed to ammonium as a nitrogen source for synthesis. Such processes may take place in lagoons or wetlands.

#### 4.2.5.2 Nitrogen Fixation

Nitrogen fixation is a process by which nitrogen gas is used for cellular synthesis by blue-green algae (Cyanophyceae) when nitrogen is the growth-limiting nutrient in an aquatic environment. Such a limitation may be observed in lakes and estuaries but is not likely to occur in MWWTP processes.

### 4.2.5.3 Chlorination

Ammonia can be removed by chlorination by being transformed into nitrogen gas (Table 4.1). As the chlorine dosage is increased, monochloramines are first formed, followed by di- and trichloramines, which are associated with a reduction in the ammonium concentration until the "breakpoint" dosage, beyond which there is no more ammonium, and free chlorine becomes measurable in solution. Alkalinity would be consumed by this reaction. Nitrite can also be converted to nitrate by chlorination.

# Chapter 5
# Ammonia Removal Processes

# Chapter 5
# Ammonia Removal Processes

This chapter presents an overview of technologies that can be applied for the removal of total ammonia-nitrogen from municipal wastewaters. These include various biological and physical/ chemical processes that can readily and consistently achieve a target effluent total ammonia-nitrogen discharge concentration of <5 mg/L. Additional details on specific process design and operational considerations, advantages and limitations, and a listing of some of the suppliers and associated contact information for the respective processes are provided in Appendix A. The treatment processes described in this chapter are listed below:

- *Suspended-growth nitrification processes*
    - Activated sludge (i.e., various designs, including biological nutrient removal process configurations)
    - Oxidation ditch
    - Sequencing batch reactor
    - Membrane bioreactor
    - Aerated lagoon (dependent on climatic conditions; seasonal nitrification only)

- *Attached-growth nitrification processes*
    - Trickling filter
    - Rotating biological contactors and submerged biological contactors
    - Aerobic submerged fixed beds (i.e., biological aerated filters)
    - Aerobic submerged mobile beds (i.e., fluidized-bed and moving-bed biofilm reactors)
    - Recirculating sand filters (lagoon effluent polishing processes)
    - Intermittent sand filters (lagoon effluent polishing processes and seasonal ammonia removal only)
    - Constructed wetlands (lagoon effluent polishing processes; seasonal ammonia removal)

- *Combined suspended-growth and attached-growth nitrification processes*
    - Dual biological processes (various process combinations)
    - Hybrid systems (e.g., integrated fixed-film activated sludge)

- *Physical/chemical processes*
    - Breakpoint chlorination
    - Air stripping
    - Ion exchange
    - Membrane separation

- *Other biological processes*
    - Facultative lagoons (climate dependent; seasonal ammonia removal only)

Various scenarios are possible in the application of ammonia removal processes for the enhanced treatment of municipal wastewaters. These scenarios and applications can include:

- new stand-alone installations, as greenfield or replacement treatment plants;
- new treatment process stage(s) for carbon oxidation and nitrification, installed downstream of an existing primary treatment plant;
- new process stage or process retrofit installed upstream of an existing lagoon to achieve year-round simultaneous carbon oxidation and nitrification;
- new add-on nitrifying posttreatment stage installed downstream of an existing non-nitrifying secondary treatment plant;

- expansion of a secondary treatment plant for increased performance and/or capacity by increasing the volume of existing process stages;
- retrofit of an existing secondary treatment plant for increased performance and/or capacity through process reconfiguration and/or the addition of new process equipment; and
- optimization of process conditions at an existing secondary treatment plant for increased performance.

Descriptions of established biological and physical/chemical processes for ammonia removal are given in Sections 5.1 and 5.2, respectively. These processes have been successfully applied at the full scale for the treatment of municipal wastewaters and are capable of achieving total ammonia-nitrogen residual concentrations of below 5 mg/L, depending on applied process conditions. The processes described do not necessarily represent a complete listing of the many possible technologies and approaches that are available for the removal of total ammonia-nitrogen and/or total nitrogen.

The main focus of this chapter is on biological nitrification processes that represent the most technically feasible and economically viable methods for ammonia removal in the treatment of municipal wastewater. Unlike physical/chemical processes, nitrification does not result in the formation of unwanted by-products. Background information on the different ammonia removal mechanisms that are possible during biological treatment is provided in Section 5.1.1. This is followed in Section 5.1.2 with an overview of possible process configurations, as well as design aspects and considerations for nitrification system applications. Only preliminary information on process design aspects is provided in this report. Other literature sources should be consulted for more comprehensive process design details and procedures (e.g., Metcalf & Eddy, 1991; Randall *et al.*, 1992; USEPA, 1993; WEF and ASCE, 1998).

An overview of the various specific nitrification processes according to suspended-growth, attached-growth and combined suspended-growth and attached-growth process classifications is provided in Sections 5.1.3, 5.1.4 and 5.1.5, respectively. Where applicable, process configurations that can allow for enhanced biological nutrient removal (EBNR) (i.e., control of effluent total nitrogen and phosphorus using biological processes) are included in the discussions in these sections. Section 5.1.6 describes other conventional biological processes that can be employed for ammonia removal, while Section 5.4 provides a brief overview of promising emerging and innovative treatment technologies. Section 5.3 provides a description of EBNR processes, with emphasis on those processes that include a nitrification process stage as an integral component.

Section 5.5 provides order-of-magnitude-level capital and O&M cost estimates for selected biological nitrification processes. The reported costs are based on vendor information, published cost equations and curves, and software programs.

## 5.1 Biological Processes for Ammonia Removal

### *5.1.1 Background on Removal Mechanisms*

Nitrification by aerobic biological treatment is the most widespread and successfully demonstrated method for achieving consistent removal of total ammonia-nitrogen from municipal wastewaters. Nitrification systems offer the potential for virtually complete and continuous removal of total ammonia-nitrogen (i.e., <1–2 mg/L) from wastewaters, depending on the applied and prevailing process conditions (USEPA, 1993).

Certain biological treatment processes that are utilized for nitrification are capable of achieving denitrification within the same reactor for the enhanced control of total nitrogen.

In the design and application of biological processes for the removal of total ammonia-nitrogen from municipal wastewater, the various forms of nitrogen must be taken into consideration. For instance, although only about 60% of the total nitrogen content of raw municipal wastewater consists of total ammonia-nitrogen (Randall *et al.*, 1992), most of the remaining nitrogen is organic and is metabolized into ammonium (i.e., through hydrolysis of urea and particulate organic nitrogen). This additional ammonium loading also has to be considered for treatment. A fraction of the available total ammonia-nitrogen is removed through assimilation to support the growth of new microorganisms that are involved in the biodegradation of carbonaceous biochemical oxygen demanding substances (cBOD) or in ammonia oxidation (i.e., nitrifying populations). The relative amount of nitrogen removed by synthesis for incorporation into new biomass depends on the yield of biological sludge and normally ranges from 8 to 20% (Randall *et al.*, 1992).

It is important to consider that in the processing of waste sludge generated at an MWWTP, contaminated sidestreams may be generated that are usually recycled to the head of the liquid train for treatment. These sidestreams can contribute significantly to organic nitrogen and total ammonia-nitrogen loading. Sludge processing steps can include thickening, digestion, dewatering, drying and incineration, all of which produce a supernatant liquor. In the typical practice of anaerobic digestion, in particular, much of the sludge-associated nitrogen becomes solubilized. The resultant high-strength supernatant (e.g., 500–1000 mg total ammonia-nitrogen/L) is usually recycled back to the liquid train, thereby increasing the total ammonia-nitrogen loading to the wastewater treatment process (CG&S and Hydromantis, Inc., 1996).

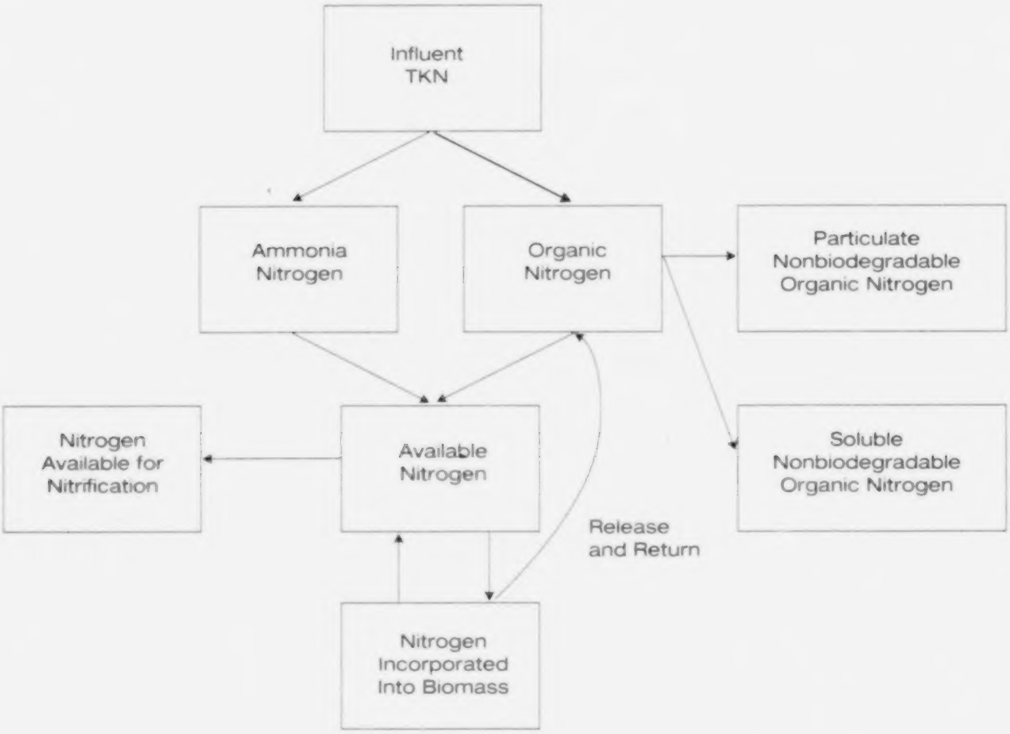
In summary, the amount of ammonia that requires removal by nitrification roughly equals raw wastewater TKN loading plus total ammonia-nitrogen loads in any recycle streams, minus:

- the amount of nitrogen that is removed with waste biomass; and
- TKN loading in the effluent discharge (i.e., allowable residuals of total ammonia-nitrogen and organic nitrogen); note that soluble non-biodegradable organic nitrogen in treated effluent discharges is typically about 0.5–2.5 mg/L (WEF, 1998; WEF and ASCE, 1998).

Figure 5.1, taken from USEPA (1989b), illustrates nitrogen metabolism and removal pathways in nitrifying systems.

Another possible mechanism for ammonia removal in biological treatment processes is volatilization. Stripping of ammonia can be particularly pronounced in low-rate facultative lagoons due to elevated pH conditions that can develop as a result of algal growth and photosynthesis reactions during warm summertime conditions. The larger fraction of total ammonia that is present in un-ionized form at conditions of elevated pH and temperature increases the potential for stripping. Uptake of total ammonia-nitrogen for algal growth can be a significant removal mechanism during summertime facultative lagoon operations.

**Figure 5.1 Nitrogen Metabolism and Removal Pathways in Nitrifying Systems (USEPA, 1989b)**


Influent TKN
Ammonia Nitrogen
Organic Nitrogen
Particulate Nonbiodegradable Organic Nitrogen
Nitrogen Available for Nitrification
Available Nitrogen
Soluble Nonbiodegradable Organic Nitrogen
Release and Return
Nitrogen Incorporated Into Biomass

### 5.1.2 Nitrification Process Configurations and Design Considerations

Various aerobic biological treatment process types, designs and configurations are available for effective nitrification of municipal wastewaters. Nitrification processes can be classified as suspended-growth, attached-growth or combined suspended- and attached-growth systems, depending on the form of the nitrifying biomass. Furthermore, process configurations can involve single-stage or separate-stage nitrification. Single-stage nitrification processes are designed for simultaneous cBOD removal by oxidative degradation and ammonia removal through assimilation and nitrification. Alternatively, separate-stage nitrification designs can be utilized involving carbon oxidation and assimilation of some of the available nitrogen in a high-rate aerobic biological reactor, followed by a dedicated lower-rate nitrification stage relying mainly on autotrophic nitrifying biomass for oxidation of the remaining ammonia.

Single-stage nitrification is the most typical process arrangement employed and represents the most feasible option for low- to medium-flow-rate applications. The more complex separate-stage configuration can provide increased stability to the relatively sensitive nitrification process, especially where the MWWTP influent is relatively concentrated and/or contains potentially inhibitory industrial waste constituents. Although the two-stage system configuration has an increased degree of complexity and O&M

requirements and produces a greater amount of sludge, it may allow for reduced overall process volumes and land area requirements, especially for larger-flow applications (WEF, 1998).

An important consideration in the application of nitrification processes is that nitrifying microorganisms possess slower growth rates, are influenced to a greater extent by influent variability and process conditions such as temperature, DO concentration, nutrient concentration, pH and alkalinity, and are more sensitive and susceptible to shock loads, inhibitory or toxic substances and upset conditions than are heterotrophic carbon-degrading microorganisms (Randall *et al.*, 1992; USEPA, 1993). Furthermore, owing to their faster rates of growth, heterotrophic cBOD-degrading microorganisms tend to outcompete nitrifying populations for available oxygen. Nitrification systems are, therefore, typically sized to ensure a long enough SRT in the aerobic reactor to allow the slow-growing nitrifying bacteria to reproduce. The process SRT, which is also referred to as sludge age or mean cell residence time, is the average length of time that the biological sludge is retained in the aeration tank of an aerobic process. Sufficient nitrification process SRT will ensure that a population of nitrifying bacteria can be developed and sustained and that the possibility of washing them out of the system is minimized. Careful control of aerobic process SRT is, therefore, a critical aspect of nitrification systems.

Nitrification requirements can lead to aerobic reactor volumes that are larger than those required for carbon oxidation. In particular, at very low process temperatures (i.e., 10°C or lower), the required process SRT and corresponding biomass inventory can be considerable, owing to the significant effect of temperature on nitrification activity (USEPA, 1993; WEF and ASCE, 1998; WERF, 1999). While minimum design SRTs for suspended-growth nitrification are only 3 days at a process temperature of 20°C, minimum required levels under wintertime cold climate conditions are longer, such as 10 days at process temperatures of 10°C and 18 days at 5°C, based on U.S. EPA models (USEPA, 1993; WEF, 1998). Seasonal variations in effluent total ammonia-nitrogen levels can occur due to the temperature dependence of nitrification. However, nitrification in suspended-growth activated sludge systems has reportedly been achieved at temperatures as low as 2°C (USEPA, 1993). Similarly, effluent total ammonia-nitrogen levels of below 1 mg/L have been achieved on a consistent basis using an attached-growth trickling filter process, even at low wastewater temperatures of 2°C, at the Reno-Sparks, Nevada, MWWTP (WEF, 1998).

The actual design aerobic SRT for a suspended-growth nitrification process is determined by multiplying the theoretical minimum SRT for ensuring nitrification by a safety factor of about 1.5–3.0 (Randall *et al.*, 1992; WEF and ASCE, 1992). This takes into account anticipated site-specific variations in influent loading and characteristics, environmental factors, uncertainties in the kinetics of the biological reactions and the potential presence of inhibitory compounds, particularly when industrial effluent contributions to the influent are significant (USEPA, 1993). It is recommended that nitrification processes be designed for conditions of non-limiting pH (i.e., 6.5–8.0 standard units) and DO concentration (i.e., >2.0 mg/L at normal loading conditions and a minimum of 0.5 mg/L under peak loading) to ensure satisfactory performance and stability (USEPA, 1993). In certain cases, supplemental alkalinity must be provided to maintain a satisfactory pH level. In total nitrogen control systems that rely on predenitrification, the design SRT must be increased in proportion to the relative volumetric fraction of the anoxic zone (e.g., typically 20–40% of the total aeration tank volume) to ensure that the target aerobic SRT level is attained.

The cBOD:TKN ratio in the influent to the biological process is a critical factor that can influence the relative fraction of nitrifying microbial populations making up the biomass in suspended-growth or attached-growth processes. Together with SRT, the carbon-to-nitrogen ratio is an important consideration in system design. Nitrifying bacteria make up only about 2–5% of the total biomass quantity in single-stage nitrification processes with typical influent cBOD:TKN ratios of 5–10, while separate-stage nitrification, with typical cBOD:TKN ratios of less than 1, can result in a nitrifier fraction of 10–35% (Metcalf & Eddy, 1991; WEF, 1998). The population density of nitrifiers and the activity of the nitrifying microorganisms influence the specific nitrification rate of the process biomass, which determines the minimum inventory of biomass required to achieve consistent ammonia removal.

In summary, the reactor volume, total active biomass inventory and oxygen transfer capacity are the principal controlling variables in nitrification processes. The minimum SRT required for nitrification depends on wastewater composition and temperature as well as on DO and pH levels in the aeration tank. The generally accepted approach for the design of suspended-growth nitrification systems is as follows (Metcalf & Eddy, 1991; Randall *et al.*, 1992; USEPA, 1993; WEF and ASCE, 1998):

1. Determine influent wastewater characteristics and select desired concentration of total ammonia-nitrogen in the treated effluent.
2. Estimate a safety factor that allows target discharge concentrations of total ammonia-nitrogen to be achieved during anticipated peak and other transient loading conditions.
3. Estimate the maximum specific growth rate (i.e., related to reaction rate for ammonia oxidation) of the nitrifying bacteria, taking into account the minimum temperature and other environmental conditions; it is assumed that the reactor design will ensure non-limiting DO and pH conditions (USEPA, 1993).
4. Determine the theoretical minimum SRT requirement based on the maximum specific growth rate estimated in (3) above.
5. Determine the design SRT, taking into account the safety factor determined in (2) above.
6. Calculate an HRT and corresponding aeration tank volume required to achieve the target total ammonia-nitrogen discharge concentration, based on an assumed design mixed liquor volatile suspended solids (MLVSS) concentration.
7. Calculate oxygen requirements and sludge production rates using appropriate mass balances and stoichiometry for nitrification. Based on the type of aeration system selected, the required air flow rates and motor horsepower can then be calculated.

The design of attached-growth nitrification systems is more complicated due to the influence of mass transport of substrates to the biofilm and associated diffusional resistances. In addition to influent characteristics and process environmental conditions, other factors must be taken into consideration in process design — namely, hydrodynamics, mass transfer characteristics and process-specific features such as support media characteristics and configurations (USEPA, 1993). Designs are usually based on an estimated surface area-specific ammonia oxidation rate. This rate is based on the amount of ammonia oxidized per unit time for a specific biomass inventory in the reactor (i.e., specific nitrification rate of biomass attached to or contained within the support media). The specific nitrification rate is dependent on environmental and growth conditions (e.g., DO concentration in the bulk liquid of at least 3 mg/L) and on influent wastewater characteristics (e.g., cBOD:TKN ratio). Based on an assumed quantity of attached-growth biomass per unit of support media surface area and an assumed safety factor as described above, the design quantity of media surface area can be calculated. Based on the specific surface area characteristic of the particular support medium being considered, the required volume of medium can be determined. For some processes, volumetric loading rate to the reactor is a design parameter.

Additional details on important nitrification process parameters and design procedures are provided elsewhere (Metcalf & Eddy, 1991; Randall *et al.*, 1992; USEPA, 1992, 1993; WEF, 1998; WEF and ASCE, 1998).

### 5.1.3 Suspended-Growth Nitrification Processes

Suspended-growth nitrification systems can be categorized as either activated sludge processes or aerated lagoon processes, with the following characteristics:

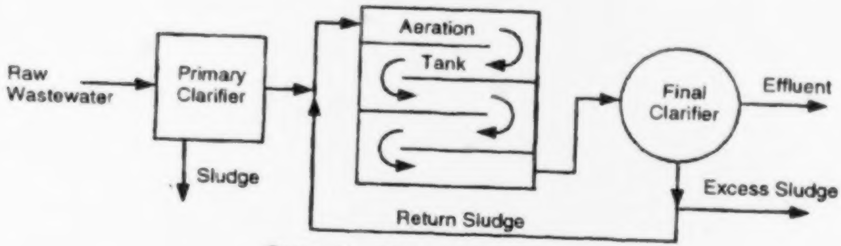
- *Activated sludge processes*
    - Activated sludge processes comprise well-mixed aeration zones with provision for the controlled retention of active suspended biomass (i.e., mixed liquor), independent of process HRT.
    - Activated sludge processes can potentially achieve virtually complete and consistent year-round ammonia removal.
    - Various traditional as well as advanced activated sludge reactor configurations exist for nitrification applications.
- *Aerated lagoon processes*
    - Aerated lagoons comprise relatively large aeration basins with surface mechanical or diffused aeration systems for oxygenation, but possess a lower degree of mixing and do not include provision for sludge recycling.
    - There is an inherently lower level of process control with aerated lagoon systems than with activated sludge processes.
    - These low-rate processes are limited to seasonal nitrification in certain locations; ammonia removal is severely reduced during cold-climate operating conditions, such as those experienced in Canada (Alberta Environment, 1987; Environment Canada, 1987; NovaTec Consultants Inc., 1996).

### 5.1.3.1 Activated Sludge Processes

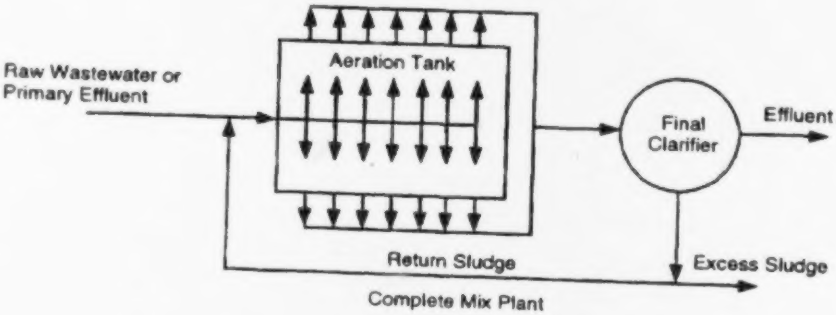
#### 5.1.3.1.1 Traditional Activated Sludge Processes

Traditional activated sludge processes are in widespread application for the treatment of municipal wastewaters (WEF and ASCE, 1998). Nitrifying activated sludge systems are available in numerous reactor designs and process configurations and can potentially produce total ammonia-nitrogen concentrations of less than 1 mg/L on a consistent basis. Variations of the traditional activated sludge process that can achieve nitrification include conventional plug flow, multiple passes in series, complete mix, complete mix tanks in series, extended aeration, contact stabilization, high-rate modified aeration and step feed configurations. Plug flow or conventional multipass designs involve aeration tank length-to-width ratios of 5–10:1 or more. The traditional continuous-flow activated sludge process typically involves the recycling of separated activated sludge from a secondary clarification stage back to the aeration tank. A schematic illustration of four traditional activated sludge process designs is shown in Figure 5.2, which has been taken from USEPA (1993).

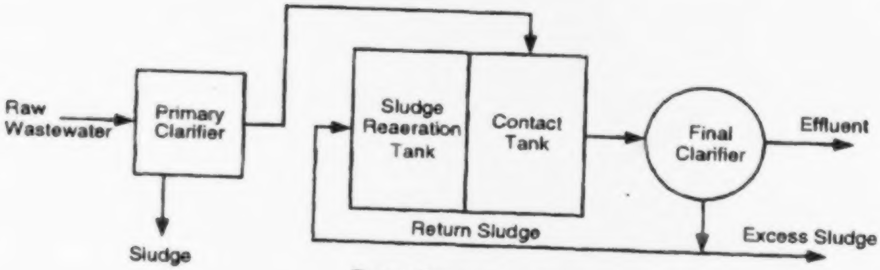
**Figure 5.2 Traditional Activated Sludge Treatment Process Configurations (USEPA, 1993)**


Raw Wastewater
Primary Clarifier
Sludge
Aeration Tank
Final Clarifier
Effluent
Excess Sludge
Return Sludge

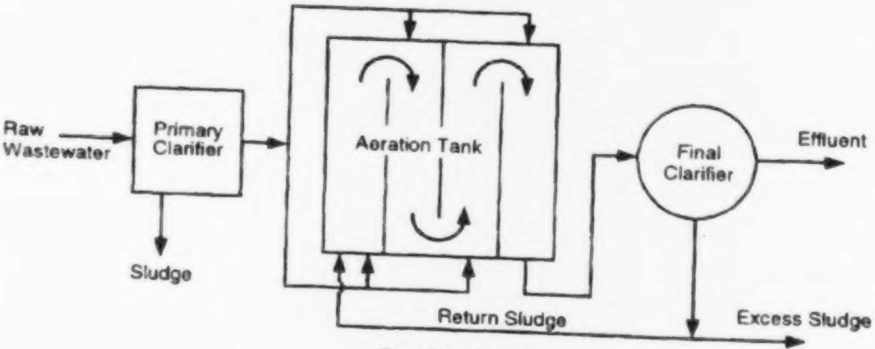
Conventional Activated Sludge Plant


Raw Wastewater or Primary Effluent
Aeration Tank
Final Clarifier
Effluent
Return Sludge
Excess Sludge

Complete Mix Plant


Raw Wastewater
Primary Clarifier
Sludge
Sludge Reaeration Tank
Contact Tank
Final Clarifier
Effluent
Return Sludge
Excess Sludge

Reaeration-Aeration Plant


Raw Wastewater
Primary Clarifier
Sludge
Aeration Tank
Final Clarifier
Effluent
Return Sludge
Excess Sludge

Step-Feed Plant

Nitrifying activated sludge processes require a long enough sludge age to ensure the retention of relatively slow growing nitrifying bacterial populations. The process is considered low to medium rate depending on applied process conditions (WEF, 1990). Oxygen requirements are typically supplied through high-efficiency aeration systems, such as fine-pore aerators for diffused fine-bubble aeration or slow-speed surface mechanical aerators, which also serve to keep the mixed liquor in suspension. Useful information on the design, O&M and economic aspects of and considerations for fine-pore aeration systems as well as the characteristics of various types of fine-pore diffusers are included in a design manual by USEPA (1989b). Information on aeration system capacity analysis and related considerations for nitrification applications is provided in CG&S and Hydromantis, Inc. (1996). Although less prevalent in municipal wastewater treatment applications, pure oxygen activated sludge processes can be employed for nitrification. These processes utilize covered and more compact aeration tanks, but are costlier to operate owing to the required supply of pure oxygen.

Typical complete mix or plug flow activated sludge processes are designed for a continuous-flow mode of operation, with HRTs of less than 12 hours in the aeration tank, and employ an external secondary clarification stage (i.e., solids sedimentation or flotation process) downstream of the aeration tank for the separation of suspended solids from the mixed liquor. The separated and thickened sludge stream is recycled back to the aeration tank (i.e., as return activated sludge, or RAS), and a resultant clarified effluent stream is either discharged or treated further (e.g., tertiary treatment and/or disinfection). Nitrifying activated sludge processes depend on the settling properties of process biomass (i.e., the mixed liquor), which helps to ensure proper control of SRT. Excess quantities of biological sludge are regularly removed from the process as waste activated sludge (WAS) for separate handling and disposal.

Nitrification can be achieved in either a single-stage (i.e., combined carbon oxidation and nitrification) or a two-stage activated sludge process. The single-stage configuration is the most prevalent for municipal wastewater treatment applications. In the two-stage process configuration, various options can be employed for initial high-rate carbon oxidation, followed by a separate lower-rate nitrification process stage. For example, a high-rate aeration activated sludge or a more compact high purity oxygen activated sludge process could be employed as the initial treatment stage. This configuration enhances the nitrification activity and stability of the downstream nitrification stage through the removal of cBOD and inhibitory substances (e.g., from industrial contributions). This maximizes the specific nitrification rate of the MLVSS in the downstream nitrification stage and allows for lower effluent total ammonia-nitrogen levels. However, the capital and O&M costs for a two-stage activated sludge system are higher than for the single-stage process for combined cBOD removal and nitrification. Additional details on the two-stage activated sludge process can be found in WEF (1990, 1998).

Extended aeration activated sludge plants are similar to the conventional activated sludge process except that they are designed for significantly longer HRTs (i.e., 18–36 hours) and SRTs (i.e., >20 days) and operate in an endogenous respiration phase, resulting in partial oxidation of biological solids (USEPA, 1992). Extended aeration processes require a higher level of oxygenation but have a lower net yield of excess sludge that requires disposal. The long SRT, low rate of operation and high degree of oxygenation associated with the process promote complete and consistent year-round nitrification (OMOE and Environment Canada, 1993). However, there is the possibility of poor solids settleability due to the formation of pinfloc at long SRTs, sludge bulking due to low F:M conditions and rising sludge due to denitrification activity in the secondary clarifier during warm periods. Furthermore, the extended aeration activated sludge process may be susceptible to freezing problems in extreme cold climates. Process designs with provision for covered aeration and clarifier tanks could be considered, if necessary.

Combined single-sludge nitrification/denitrification process configurations are often employed for the enhanced control of effluent total nitrogen. These systems usually employ up-front anoxic zones representing 20–40% of the total aeration tank volume (i.e., anoxic plus aerobic zones) for predenitrification. In addition to the RAS stream, mixed liquor from the aerobic zone is typically recycled to the anoxic zone at rates of 2–4 times the influent flow rate for the enhanced treatment of nitrates. Effective predenitrification minimizes the potential for generation of nitrogen gas and floating sludge in the secondary clarification stage, which enhances SRT control.

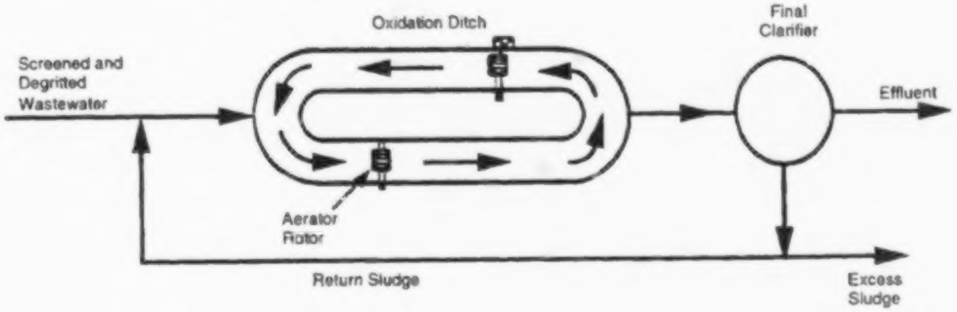
More elaborate activated sludge process configurations are possible, either as greenfield installations or as process retrofits, that are capable of biological nutrient removal. In addition to nitrification and denitrification, biological nutrient removal processes can achieve enhanced biological phosphorus removal (EBPR) through the use of an up-front anaerobic selector zone. As a result, chemical additions can be eliminated and excess sludge generation rates reduced. Randall *et al.* (1992) provide several examples of how existing conventional activated sludge-based MWWTPs have been successfully retrofitted into biological nutrient removal processes. This can often be accomplished with little or no expansion of the existing process, such as the Step Bio-P process upgrade at the MWWTP in Lethbridge, Alberta, which was retrofitted to include several distinct process zones but without any additional tankage requirements.

### 5.1.3.1.2 Oxidation Ditch

The oxidation ditch technology is an activated sludge-type process that is typically operated in extended aeration mode. It consists of a simple racetrack aeration tank and is usually coupled with an external secondary clarifier. Degritted and screened wastewater is introduced into the anaerobic zone of the closed-loop oval channel reactor. Downstream of this inlet location, the mixed liquor becomes aerated by means of a rotating brush aerator or other aeration device, which imparts a velocity onto the mixed liquor, causing it to recirculate through the aeration tank at about 0.25–0.35 m/s (Metcalf & Eddy, 1991). While nitrification activity can occur in the aerobic zone, the mixed liquor becomes oxygen deficient upstream of aerators providing anoxic zones for denitrification. The aerobic, anoxic and anaerobic zones that can develop are not physically separated within the reactor. A simple process layout is shown in Figure 5.3, taken from USEPA (1993).

With adequate aeration capacity, nitrification can be readily achieved using the oxidation ditch process, which is considered to be stable and reliable (USEPA, 1992). The single-sludge process can achieve simultaneous nitrification and denitrification, resulting in total nitrogen removals of 40–80%. The rates of both nitrification and denitrification are inherently low. However, the large inventory of mixed liquor in the system compensates for these low reaction rates (WEF and ASCE, 1992).

**Figure 5.3 Schematic of a Typical Oxidation Ditch Process (USEPA, 1993)**



Biological nutrient removal operations are possible through the proper control of oxygen addition and can be enhanced by the installation of an upstream anaerobic reactor for contacting raw wastewater with a recycled mixed liquor stream (WEF, 1998).

#### 5.1.3.1.3 Sequencing Batch Reactor

The SBR technology is another variation of the activated sludge process. However, in contrast to the continuous-flow mode of traditional activated sludge process operation, SBRs operate in a discontinuous, variable-volume fill-and-draw mode. The process biomass cycles through alternating anoxic, anaerobic and aerobic conditions in the biological reactor. Furthermore, biological treatment and sludge sedimentation and decanting occur in the same basin but in alternating sequences involving complete mix conditions during the react phase and quiescent conditions during the sedimentation phase. This eliminates the requirement for an external secondary clarification stage and RAS stream. Air for the nitrification process is supplied through fine-bubble membrane diffusers, coarse-bubble aerators or a jet aeration system. The sequential operating phases of the SBR process are listed in Table 5.1 and depicted in Figure 5.4 (Hydromantis, Inc. and SBR Technologies Inc., 1998).

As with any suspended-growth nitrification system, an appropriate aerobic SRT must be designed for. Accordingly, consideration is given only to the relative time that the mixed liquor is maintained in aerobic mode.

#### 5.1.3.1.4 Membrane Bioreactor

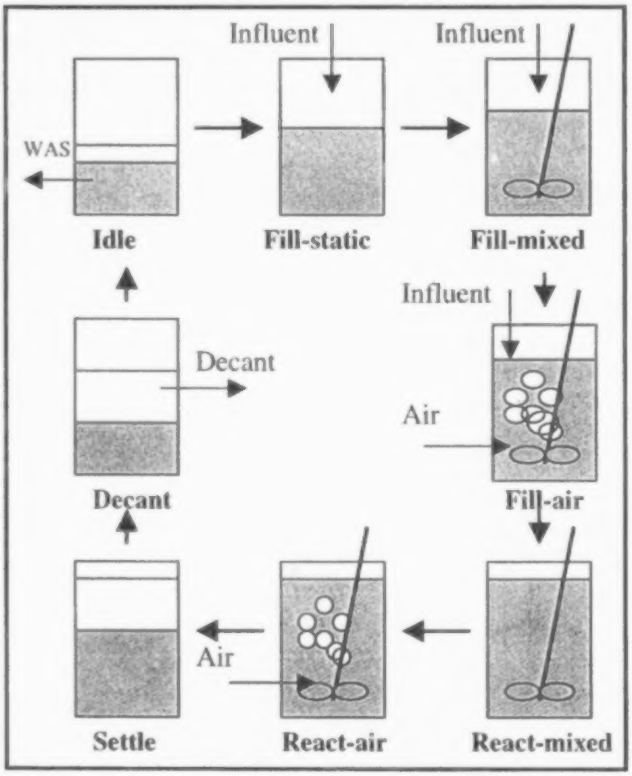
The MBR technology represents an advanced activated sludge wastewater treatment process that is capable of producing a suspended solids-free, tertiary-quality effluent. The process combines a biological reactor with a membrane filtration system for retention of the activated sludge. The most common MBR process configuration utilized for the treatment of domestic wastewaters involves direct filtration of mixed liquor by immersed microfiltration or ultrafiltration membranes (Trussell *et al.*, 2000). The process eliminates the need for an external solid/liquid separation stage, such as secondary clarification with the RAS system, and filtration. Hollow fibres and flat sheet membranes have been employed in this *in situ* filtration configuration. Both designs rely on dedicated aeration systems below the membrane modules for membrane surface

**Table 5.1 Sequencing Batch Reactor Operating Phases**

| Phase | Operations |
|---|---|
| 1. Static fill | Influent introduced to idle basin |
| 2. Mixed/react fill | Influent flow continues while mixing by aeration initiated |
| 3. React | Influent flow stops while aeration and mixing continue |
| 4. Settle | Aeration and mixing stopped and clarification commences |
| 5. Decant | Clarified supernatant is decanted |
| 6. Idle | Basin on standby, excess waste sludge is removed, and reactor is awaiting refilling and a repeat of phases |

**Figure 5.4 Schematic of a Typical SBR Process Cycle**

(Hydromantis, Inc. and SBR Technologies Inc., 1998)


Influent
Influent
WAS
Idle
Fill-static
Fill-mixed
Influent
Decant
Air
Decant
Fill-air
Air
Settle
React-air
React-mixed

scouring and control of fouling and filtrate flux. Air required to sustain the biological nitrification process is supplied separately through fine-bubble diffusers. Maintenance of an acceptable filtrate flux requires cleaning of the membranes with chemicals. This involves regular in-place cleaning and occasional more intensive and extensive cleaning of the membranes in an external cleaning tank (WERF, 2000).

Very high mixed liquor solids concentrations of 10 000–20 000 mg/L can be maintained in the MBR process with immersed membranes. Unlike conventional activated sludge systems, solid/liquid separation in the MBR process does not rely on biomass settling properties. The concentrated activated sludge permits elevated design loading rates and compact MBR process designs. The process is typically operated at long SRTs (i.e., >15 days) and low F:M ratios to enhance filtration performance, so nitrification is typically readily achieved even under cold-climate operating conditions. Immersed membrane-based processes typically involve recirculation of mixed liquor from below the membrane modules to the inlet area of the bioreactor to prevent difficulties with biomass buildup and clogging (Crawford *et al.*, 2000). Directing this recycle flow to a preanoxic zone enables total nitrogen control and recovery of alkalinity through denitrification.

The MBR process with the most widespread application in North America for the treatment of domestic wastewaters is the proprietary ZenoGem® process developed and marketed by Zenon Environmental Inc., with over 100 installations for wastewater flow rates ranging mainly between 10 and 200 $m^3$/d (WERF, 2000). A number of applications with higher wastewater flow rates of between 1000 and 7600 $m^3$/d have recently been installed (i.e., since 1997), the largest being in Powell River, B.C. (Crawford *et al.*, 2000). Designs have been prepared for flow rates of up to 40 000 $m^3$/d (8.8 MIGD), and construction is under way for this large ZenoGem application in Europe.

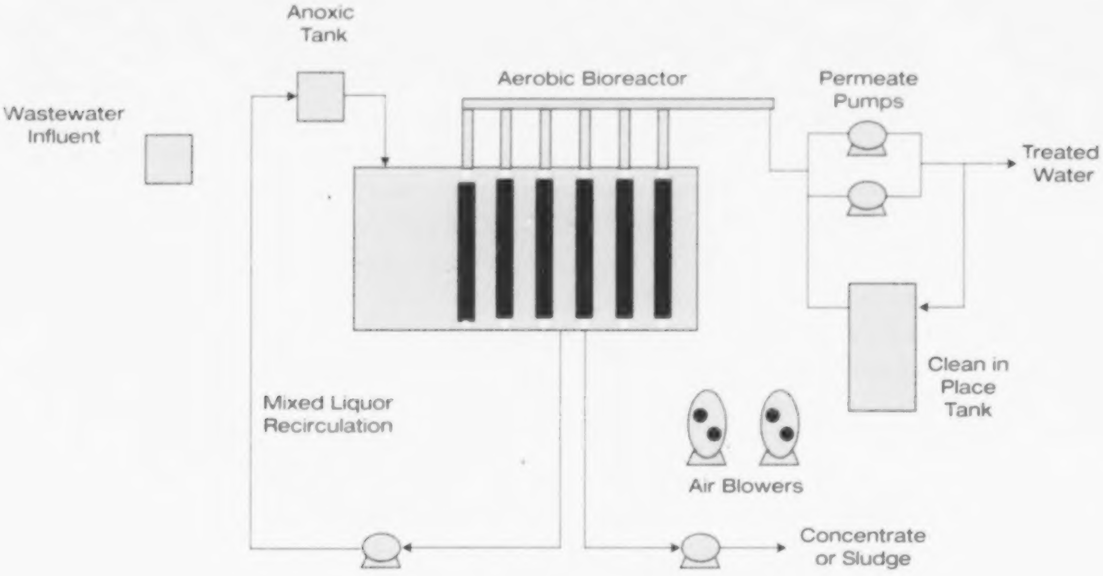
The ZenoGem process utilizes hollow fibre membranes that are loosely held between module support frames with integral coarse-bubble aerators and permeate withdrawal lines. The process is driven by a slight vacuum using permeate pumps to draw treated effluent into the hollow fibres, which are immersed in the mixed liquor. A filtrate flux of 25 L/$m^2$ per hour is typically used as an average design level. Figure 5.5 shows a schematic of a typical ZenoGem plant (i.e., modification of a schematic from Zenon Environmental Inc.).

Additional details on MBR process design and operational considerations and experiences are described in recently reported literature (Crawford *et al.*, 2000; Fernandez *et al.*, 2000; Trussell *et al.*, 2000; WEF, 2000a; WERF, 2000).

#### 5.1.3.2 Aerated Lagoon Processes

Aerated lagoons and aerated facultative lagoons are low-rate suspended-growth processes that utilize mechanical systems for aeration and are typically operated in a continuous-discharge mode. Aerated lagoon-based MWWTPs offer the potential for nitrification and effective ammonia control under ideal conditions, including warm process temperatures and relatively long HRTs in the aerated cells (USEPA, 1993). However, aerated lagoons and aerated facultative lagoons are susceptible to seasonal effects in cold-climate applications that are typical in Canada, resulting in significantly lower ammonia removal (OMOE and Environment Canada, 1993; NovaTec Consultants Inc., 1996). Consequently, effluent total ammonia-nitrogen concentrations exceeding 5 mg/L have been reported for continuous-discharge aerated lagoon and aerated facultative lagoon systems in Canada during wintertime and

**Figure 5.5 Schematic of an Immersed Membrane-Based MBR Treatment Process**



springtime (OMOE and Environment Canada, 1993; NovaTec Consultants Inc., 1996). Therefore, to consistently achieve target effluent discharge ammonia concentrations of less than 5 mg/L in cold-climate applications, if necessary, provision must be made for an add-on tertiary treatment stage for year-round nitrification in continuous-discharge mode or for long-term effluent storage and polishing permitting controlled annual discharge in autumn.

#### 5.1.3.2.1 Aerated Lagoons

Aerated lagoon wastewater treatment processes comprise one or more treatment cells that are oxygenated using mechanical devices such as surface mechanical aerators or diffused air supply systems. A quiescent zone is provided in the vicinity of the aerated lagoon outlet to allow for settling of suspended solids. Aerated lagoons are smaller and deeper than conventional facultative lagoons but are aerated and mixed to a greater extent and offer a higher degree of process control. Aerated lagoon cells can be operated either under complete mix conditions, which enhances nitrification, or with only partial mixing, whereby some settling of suspended solids occurs. Primary treatment is not required upstream of aerated lagoon processes; however, preliminary treatment is typically employed.

#### 5.1.3.2.2 Aerated Facultative Lagoons

Aerated facultative lagoons typically consist of a series of mechanically aerated, incompletely mixed cells wherein the biodegradation of most of the wastewater organics occurs, followed by a facultative polishing cell for settling of suspended solids and additional effluent treatment. Reaction mechanisms and interactions within facultative lagoons are more complex than in completely mixed aerated lagoons due to the presence of a benthic zone consisting of settled sludge deposits that undergo anaerobic decomposition, particularly during warm temperature conditions. This can lead to the solubilization and release of total ammonia-nitrogen during springtime, following cold wintertime operations with reduced sludge activity and extended periods of ice cover. Unlike conventional facultative lagoons, which rely on photosynthesis by algae and some surface reaeration for the supply of oxygen to support biodegradation, aerated facultative lagoons rely largely on mechanical aeration for the supply of oxygen. Primary treatment is not required upstream of aerated facultative lagoon processes.

### *5.1.4 Attached-Growth Nitrification Processes*

Attached-growth nitrification processes can typically produce year-round effluent ammonia concentrations of 1–3 mg/L (Metcalf & Eddy, 1991). They can include either single-stage biological treatment for combined carbon and ammonia oxidation or a two-stage arrangement involving a separate downstream nitrification stage (i.e., tertiary nitrification, where the influent cBOD$_5$:TKN ratio is typically <1.0 and the soluble BOD$_5$ concentration is <12 mg/L) (WEF and ASCE, 1992, 1998; USEPA, 1993). The initial high-rate carbon oxidation stage preceding an attached-growth tertiary nitrification stage can involve either suspended-growth or attached-growth secondary treatment.

Attached-growth processes are biological treatment systems that rely on microorganisms that are attached to inert support material. The microorganisms developed on the surface of the media are referred to as a biofilm.

Attached-growth systems are traditionally known for their capacity to accommodate short periods of peak load variations and their higher resistance to toxicity events. Due to the physical attachment of the microorganisms to the media, the biomass is less susceptible to microorganism flush during hydraulic peaks and low-nutrient periods, compared with suspended-growth biomass. The biological conversion and oxidation of organic matter and ammonia by the biomass rely on diffusion inside the biofilm. In the case of nitrification, the diffusion of ammonia and oxygen into the biofilm is particularly important. Thus, the contact efficiency between wastewater components, available oxygen and the biofilm surface must be favoured.

A number of attached-growth processes may be applied to achieve ammonia removal in single- or two-stage combinations. Some of these processes have many installations that have operated for several years, and others are emerging technologies with significant potential for efficient nitrification. This section covers up-to-date attached-growth technologies that have shown consistency and success in achieving nitrification. These technologies include trickling filters, RBCs, aerobic fixed-bed reactors (BAF), including Biofor and Biostyr processes, aerobic mobile-bed reactors, including fluidized beds and moving-bed biofilm reactor (MBBR), recirculating and intermittent sand filters and constructed wetlands. Additional details on process characteristics related to the respective technologies described in this section are provided in Appendix A.

#### 5.1.4.1 Trickling Filters

The trickling filter process is an aerobic attached-growth reactor in which a fixed solid medium is used to support the growth of an active biological film. The primary settled and/or screened wastewater is pumped and distributed on the top surface of the medium coated with biological growth through which the water percolates downward to the underdrain. Removal of suspended and soluble organic matter and TKN occurs by sorption and aerobic assimilation by the biofilm. The oxygen requirement for the aerobic treatment is provided from natural draft or air-forced circulation fans through the interstices of the filter medium. Growth of the biofilm increases as the influent organic load and strength increase and until a maximum effective thickness is reached. The excess biofilm growth is sloughed off during filter operation on a continuous or periodic basis. The treated effluent from the trickling filter contains solids with good settling characteristics, and final clarification of the filter effluent is required to settle and collect the sloughed biomass from the medium prior to effluent disinfection or final discharge.

Trickling filters have been used for $BOD_5$ removal, $BOD_5$ removal and nitrification, and tertiary nitrification. In the case of combined $BOD_5$ removal and nitrification, referred to as a single-stage nitrification process, one trickling filter is typically used with various filter effluent recirculation patterns to satisfy wetting media efficiency and optimal hydraulic and volumetric load rates. Tertiary nitrification, referred to as two-stage, involves a secondary-treated effluent with low $BOD_5$ content and high total ammonia-nitrogen. The secondary effluent may originate from a previous trickling filter or other biological treatment, such as rotating biological reactor or activated sludge. A two-stage trickling filter with an intermediate clarification unit permits high $BOD_5$ loading rates for influent contaminant removal in the first filter and produces a clarified low $BOD_5$ and high total ammonia-nitrogen influent for tertiary nitrification in the second filter. Recirculation of the first and second trickling filter effluents is used for wetting and load rate optimal efficiencies. Figure 5.6 presents a schematic of single-stage and two-stage trickling filters with typical recirculation lines.

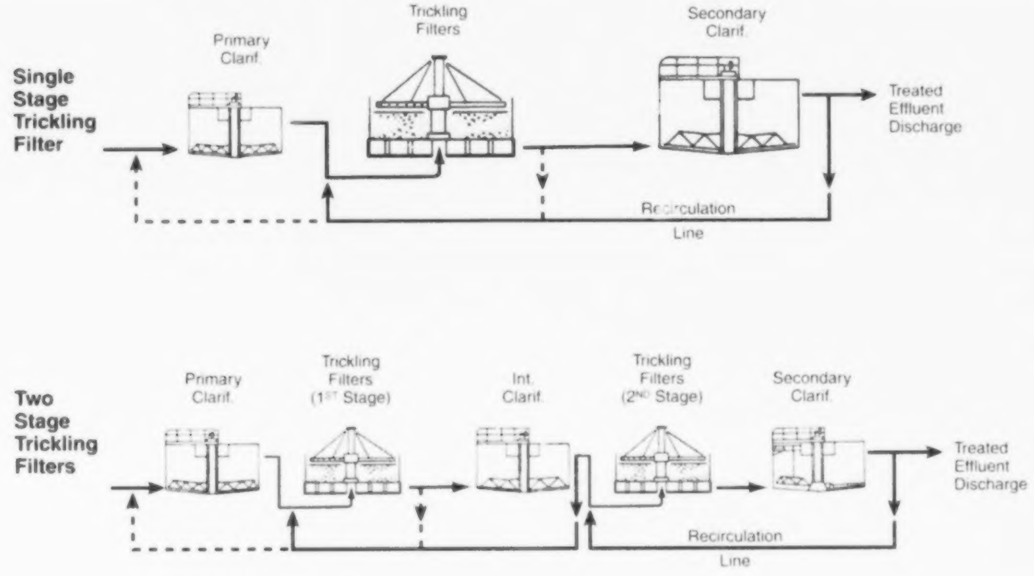
The hydraulic component of the trickling filter is comparable to a vertical plug flow; consequently, the biomass growth through the depth of the filter is a function of the substrate available. The biofilm developed on the medium of the tricking filters is typically stratified with the fast-growing heterotrophic organisms in the upper zone of the filter associated with $BOD_5$ removal and the slow-growing autotrophic nitrifiers in the lower zone of the filter associated with low $BOD_5$ levels and nitrification activity (WEF, 2000b).

The nitrification process in the trickling filter system is sensitive to nutrient diffusion efficiency in the layer of biofilm covering the medium, $cBOD_5$:TKN ratio, level of total and soluble influent $BOD_5$, oxygen availability, total ammonia-nitrogen concentration and temperature. The rate of nitrification is influenced by the type of medium used, the hydraulic and organic loading rates, and ventilation.

#### 5.1.4.2 Rotating Biological Contactors

The RBC is a treatment system that takes advantage of aerobic biofilm attached growth. The primary settled and/or screened influent is evenly distributed along the inlet side of the first RBC unit(s), as shown in Figure 5.7. The influent then enters the first pass of the RBC system. Each biological contactor consists of a cylindrical synthetic media bundle mounted on a horizontal rotating shaft partially submerged (~40% submergence) in a holding tank (USEPA, 1993; WEF, 2000b). According to the level of treatment required, the RBC system may include in one train a series of rotating contactor units separated from each other by "passes" with underflow baffles. The influent then flows from the first pass to the following passes by circulating under the dividing baffles. A biological film is developed along the train on the synthetic medium and achieves the level of aerobic treatment required to remove the contaminant load. The excess biofilm growth is sloughed off during operation on a continuous or periodic basis. The treated effluent from the RBC train contains suspended solids that

**Figure 5.6 Single-Stage and Two-Stage Trickling Filter Process Flow Configurations (Envirex)**


Single Stage Trickling Filter
Primary Clarif.
Trickling Filters
Secondary Clarif.
Treated Effluent Discharge
Recirculation Line
Two Stage Trickling Filters
Primary Clarif.
Trickling Filters (1ST Stage)
Int. Clarif.
Trickling Filters (2ND Stage)
Secondary Clarif.
Treated Effluent Discharge
Recirculation Line

are further settled in a final clarifier. The clarified effluent is then further treated or discharged.

According to the flow rate and contaminant load to be treated, the RBC system may be divided into one or several trains, with each train including a series of passes with one or more RBCs per pass. Thus, the hydraulic component of the RBC system is comparable to a plug flow.

The cylindrical medium is continuously rotated to alternately expose the biofilm to the wastewater and the atmosphere. This rotation allows adsorption of solids and diffusion of soluble $BOD_5$, ammonium-nitrogen and other nutrients and oxygen transfer into the biofilm for the aerobic treatment. Organic components and ammonium-nitrogen are then oxidized and synthesized in new cell biomass. A typical section of an RBC unit is shown in Figure 5.8. The rotation of the biological contactor unit is typically 1–1.6 revolutions per minute (USEPA, 1993; WEF, 2000b).

Rotating contactor shaft rotation is typically achieved with mechanical drive, while air drive may also be used. The mechanical drive provides a nearly constant shaft revolution speed, but the torque requirement is a function of the biofilm weight. The air drive provides a nearly constant torque requirement, but the shaft revolution speed is a function of the biofilm weight.

RBCs have been used for $BOD_5$ removal, combined $BOD_5$ removal and nitrification, and tertiary nitrification. In the case of combined $BOD_5$ removal and nitrification, referred to as a single-stage nitrification process, a series of rotating contactor passes is provided to sequentially achieve the organic load removal in the first passes and ammonia removal with nitrification in the last passes. Tertiary nitrification, referred to as two-stage, involves a

**Figure 5.7 Single-Stage Rotating Biological Contactor Diagram (USEPA, 1993)**

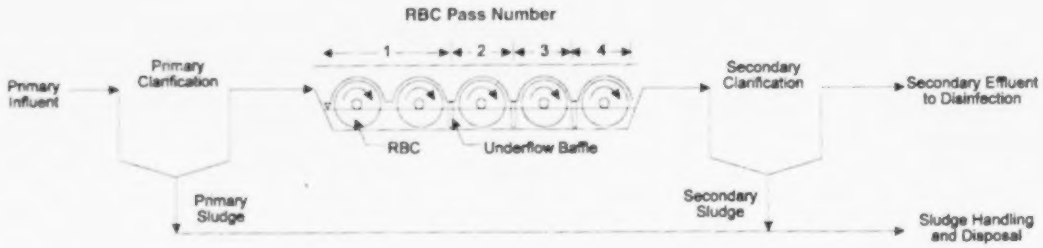
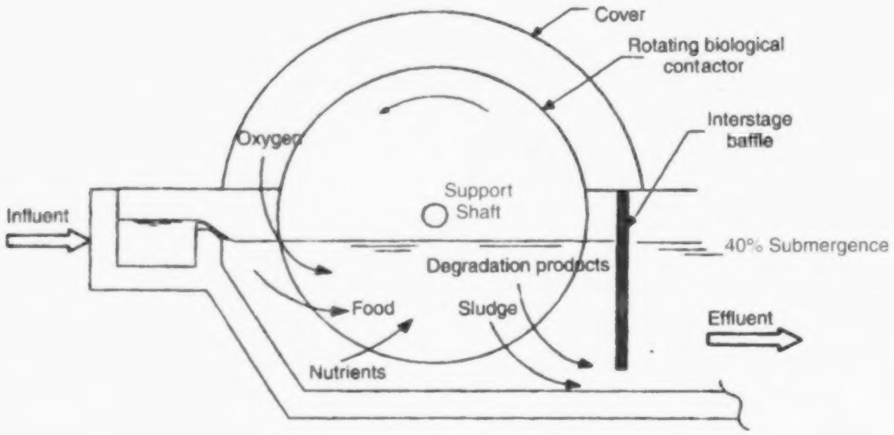

RBC Pass Number
1
2
3
4
Primary Influent
Primary Clarification
Secondary Clarification
Secondary Effluent to Disinfection
RBC
Underflow Baffle
Primary Sludge
Secondary Sludge
Sludge Handling and Disposal

**Figure 5.8 Rotating Biological Contactor Section (WEF, 2000b)**


Cover
Rotating biological contactor
Interstage baffle
Oxygen
Support Shaft
Influent
40% Submergence
Degradation products
Food
Sludge
Effluent
Nutrients

secondary-treated effluent with low $BOD_5$ content and high total ammonia-nitrogen. The secondary effluent may originate from a previous biological treatment such as activated sludge or trickling filter. A two-stage system with an intermediate clarification prior to the RBCs permits high $BOD_5$ loading rates for wastewater contaminant removal in the first biological stage and generates a clarified effluent with low $BOD_5$ and high total ammonia-nitrogen for tertiary nitrification in the RBCs.

As for other attached-growth systems, the nitrification process in the RBC system is sensitive to nutrient diffusion efficiency in the layer of biofilm covering the medium, $cBOD_5$:TKN ratio, level of total and soluble influent $BOD_5$, oxygen availability, total ammonia-nitrogen concentration and temperature. The degree of nitrification is influenced by the hydraulic and organic loading rates.

Submerged biological contactors (SBCs) are an alternative approach to the classic RBC.

The SBC involves the same type of RBC, but the submergence is increased up to 70–90% (WEF, 2000b). The main advantage is to reduce weight loads on the shaft. The SBC typically uses a dual air header for excess biofilm growth sloughing and additional process air provision. A rotating contactor shaft air drive system is typically provided. A typical section of an SBC unit is shown in Figure 5.9 (figure provided by Envirex). This section illustrates an air drive system using diffused air and an array of cups fixed on the external side of the contactor unit to capture the diffused air and create buoyancy to provoke the rotating movement. The SBC treatment design approach is the same as for the RBC.

### 5.1.4.3 Aerobic Submerged Fixed-Bed Reactors

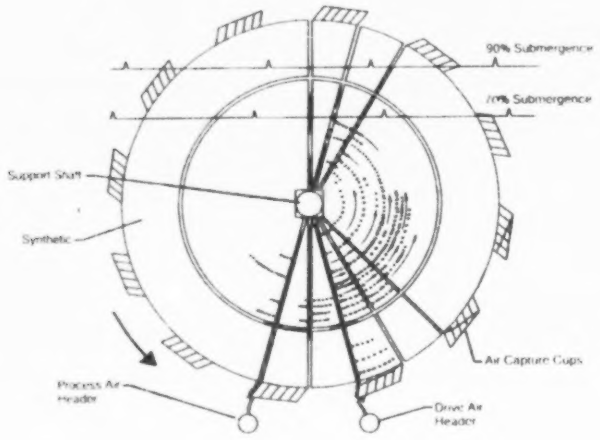
Aerobic submerged fixed-bed reactors were previously referred to as biological aerated filters, or BAF. The term fixed bed refers to the use of a non-moving medium submerged in a vertical mechanically aerated cell. Submerged fixed-bed reactors are further classified into submerged settled fixed bed and submerged floating bed according to the media density of the process (WEF, 2000b). The fine media sizes with high specific surface area used in this type of reactor offer the possibility to simultaneously achieve biological treatment and suspended solids filtration. The requirement for secondary or intermediate clarification can be eliminated. The volumetric organic loading rates (kg COD or $BOD_5/m^3$ per day) applicable to these reactors are reported to be as high as 10 times the loading values indicated for trickling filters. Thus, the reactors are more compact, and the overall site requirements and footprint may then be significantly lower. The reactors could be covered and installed in sensitive areas.

In general, studies have shown that aerobic submerged fixed-bed reactors with an organic loading rate less than 10 kg $COD/m^3$ per day for carbonaceous removal of a wastewater with a $COD:BOD_5$ ratio between 2 and 2.5 may produce a treated effluent $BOD_5$ lower than 30 mg/L (WEF, 2000b). The reported maximum organic loading rate to maintain high nitrification efficiency in a combined carbon and ammonia removal configuration (i.e., single-stage process) is 2 kg $COD/m^3$ per day, based on the volume of the medium (WEF, 2000b). Treated effluent recirculation may be provided in some cases to maintain a minimum or favour a constant filtration velocity through the medium. An interesting feature of these modular biofilters is that they could be installed in a series of trains and then be switched from series to parallel treatment when high hydraulic flow occurs to permit treatment and filtration of the total hydraulic load.

Submerged settled fixed-bed reactors include a high-density medium supported by a structural floor. The Biofor process, manufactured by Degrémont, is an application of this type of reactor. Submerged floating-bed reactors include a low-density floating medium confined in the reactor by a structural ceiling. The Biostyr process, manufactured by OTV, is an application of this reactor.

Historically, two hydraulic configurations have been installed with the submerged settled fixed-bed reactors, upflow and downflow. Downflow configurations included process trade names such as Biocarbone and Biodrof. These designs involved influent distribution on top of the medium and a downflow percolation. These flow schemes were shown to generate fast and severe clogging of the medium, which then resulted in an increase in the backwashing frequency to clean and unclog the medium. From these experiences, this type of hydraulic configuration is not recommended. Upflow hydraulic configuration,

## Figure 5.9 Submerged Biological Contactor Section (Envirex)



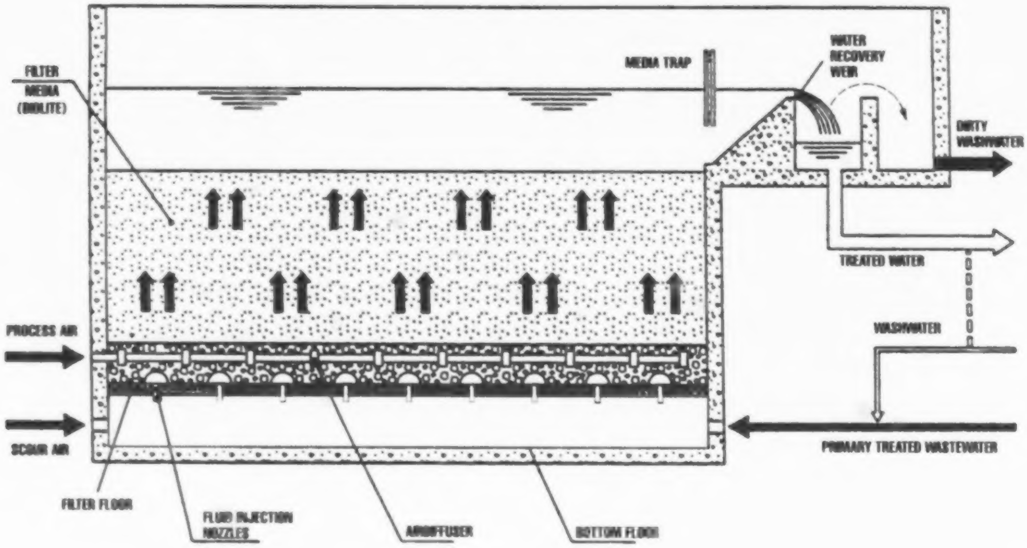
as for the Biofor and Biostyr processes, is now generally installed to favour lower-rate headloss buildup and avoid high backwashing frequency.

#### 5.1.4.3.1 Biofor Process

The Biofor process is an aerated submerged settled fixed-bed reactor with an upflow configuration and a co-current air admission. Figure 5.10 provides a schematic view of the reactor components. The Biofor units are rectangular concrete basins, with each cell comprising the following:

- two vertical sections separated by a structural filter floor:
  - the bottom section under the filter floor provides inlet to injected fluids, including primary treated wastewater, backwash water and scouring air;
  - the above section is filled with the medium in which biological reaction and filtration occur;
- a structural filter floor to support the granular medium and equally distribute the upward flow through a fluid injection system covering the surface of the filter floor;
- a fluid injection system composed of long-stem nozzles installed in the filter floor and permitting the passage of fluids injected under the filter floor;
- air headers and medium-size bubble diffusers located just above the filter floor between the rows of nozzle heads to ensure injection and equal distribution of process air through the medium for biological activity oxygen requirements;
- one front-mounted water recovery weir;
- one downstream-sloping treated water collection trough and one backwash water recovery trough; and
- one media trap located in front of the water recovery weir to reduce turbulence and media losses.

**Figure 5.10 Biofor Upflow Reactor (Ondeo Degremont)**



The Biofor system includes two sets of air blowers, one for process air and one for air scouring, and one set of treated water backwash pumps.

The wastewater flow scheme through the Biofor system first involves screening, primary settling and degreasing, if required. The primary treated wastewater is then divided among the Biofor units in a splitting chamber. The wastewater then enters the bottom section under each filter floor and is injected in the medium co-currently with process air, which promotes media expansion. The surface area of the medium supports the development of an attached-growth biomass for wastewater treatment. The medium permits the suspended solids to be retained and acts as a filtration unit. The treated water above the filter crosses the front-mounted water recovery weir and is evacuated through the water collection trough to disinfection or final discharge. A portion of the treated water is kept in a washwater tank for backwashing of the media.

Gradual headloss increases due to excess biomass growth and the retained suspended solids will reach a preset water level, at which point a backwashing event is initiated. Backwashing of the medium permits the sloughing of the excess biomass growth and the solids trapped in the filter medium.

The Biofor process is promoted to be used as a single-stage system with combined BOD$_5$ removal and nitrification and a two-stage system with separated BOD$_5$ removal and nitrification. In the case of the single-stage process, nitrification efficiency may not be as high as for a two-stage system. This has to be addressed according to the level of ammonia removal required, or pilot trials may have to be conducted. Tertiary nitrification with a two-stage system involves a secondary-treated effluent with low BOD$_5$ content and high total ammonia-nitrogen. A secondary effluent with such characteristics may originate from a previous Biofor reactor or other biological treatment system, such as aerated lagoons.

### 5.1.4.3.2 Biostyr Process

The Biostyr process is an aerated submerged bed reactor with an upflow filtration mode through a floating fine medium with co-current air diffusion. Figure 5.11 presents a schematic of the Biostyr process. The Biostyr system consists of cubical cells, with each cell including the following:

- two vertical sections separated by a structural filter ceiling:
  - the bottom section under the filter ceiling provides space for the floating medium depth and a lower medium-free zone for the inlet of the fluids, including primary treated wastewater and process/scouring air;
  - the above section is filled with the treated/filtered water and acts as a storage zone for gravity backwash;
- a structural filter ceiling with regularly spaced filtration nozzles used to retain the filter medium and to evacuate the treated water;
- a bottom feed pipe distribution system for the primary treated wastewater;
- an air header network located at the bottom of the reactor to favour the coarse-bubble distribution and aerobic conditions of the filter for biological activity;
- automatic valves in the upper section for treated water outlet; and
- automatic drain valves for backwash water recovery.

The Biostyr system uses the same set of air blowers for process air and scouring air. An automatic control system is used on the blowers to adjust the scouring air requirement for the backwash event. The Biostyr process does not use treated water backwash pumps.

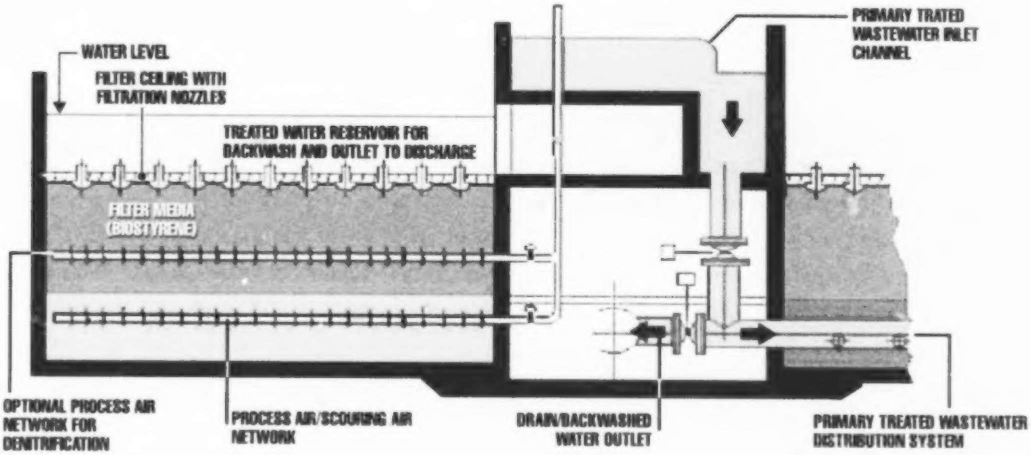
An additional feature of the Biostyr process is the possible installation of a second air header network located inside the medium depth to favour a denitrification anoxic zone under the air header.

The wastewater flow scheme through the Biostyr system requires raw wastewater screening and primary settling. The primary-treated wastewater enters an inlet channel, where it is evenly divided among the Biostyr cells. The influent is distributed on the bottom of the filter and forced up through the filter medium by the higher-elevation head at the inlet co-currently with the process air. The wastewater and process air pass through the floating medium. An attached-growth biomass develops on the medium surface for wastewater treatment. Filtration of the suspended solids by the medium grains is concomitantly achieved as the flow moves upward through the filter. The nozzles in the ceiling floor allow the treated water to enter a common water reservoir above the filters. The treated water is evacuated for further treatment or final discharge.

The excess biomass growth and the suspended solids retained within the filter medium increase the headloss on the filter through the treatment cycle. Once a preset water level is reached, a periodic backwashing event is initiated. The backwashing permits the sloughing of the excess biomass growth and the evacuation of the filtered solids.

The Biostyr process was originally developed for ammonia removal in tertiary treatment. The technology is promoted as being efficient for ammonia removal in a single-stage process for combined cBOD removal and nitrification. The nitrification efficiency for ammonia removal

**Figure 5.11 Biostyr Upflow Reactor (Kruger)**


PRIMARY TRATED WASTEWATER INLET CHANNEL
WATER LEVEL
FILTER CEILING WITH FILTRATION NOZZLES
TREATED WATER RESERVOIR FOR BACKWASH AND OUTLET TO DISCHARGE
FILTER MEDIA (BIOSTYRENE)
OPTIONAL PROCESS AIR NETWORK FOR DENITRIFICATION
PROCESS AIR/SCOURING AIR NETWORK
DRAIN/BACKWASHED WATER OUTLET
PRIMARY TREATED WASTEWATER DISTRIBUTION SYSTEM

under a single-stage mode has to be addressed in accordance with the effluent ammonia requirement, and pilot testing should be conducted to verify the level of treatment achieved under this configuration. The nitrification efficiency varies with the raw wastewater total ammonia-nitrogen concentration and the cBOD:TKN ratio.

### 5.1.4.4 Aerobic Submerged Mobile-Bed Reactors

Aerobic submerged mobile-bed reactors involve the use of particles or lightweight media put in suspension and movement with air or water flow (WEF, 2000b). Mechanical devices, such as mixers, may be used for suspension in a non-aerated environment to achieve denitrification. Attached-growth biomass is developed on the particles and media, and in some cases the particles may become relatively heavy. The water and airflow velocities are used to suspend or fluidize the particles and the media. Mobile-bed reactors are further classified into fluidized-bed and moving-bed reactors according to the media density of the process.

#### 5.1.4.4.1 Fluidized-Bed Reactors

Fluidized-bed reactors can be used as single-stage systems, for combined carbon oxidation and ammonia removal, and for tertiary nitrification in a two-stage system. They are typically aerobic systems with an upflow configuration mode and a co-current process air admission. Historically, full-scale fluidized-bed reactors have been particularly applied in industrial wastewater treatment. The fluidized media could be sand, clay or activated, powdered and granular media with a reported average dimension of 0.5 mm (WEF, 1998). The medium in suspension offers a high specific surface area for the development of attached-growth biomass, which allows for total ammonia-nitrogen loading rates higher than with traditional attached-growth technologies.

Fluidized-bed reactor configurations generally involve primary treatment followed by an external oxygen saturation of the primary-treated effluent with a recirculated portion of the final treated effluent. The total oxygenated influent enters the reactor as an upflow that promotes uniform expansion and fluidization of the medium.

Aerobic biological treatment for cBOD removal and nitrification can be achieved in the fluidized-bed reactor. The reactor effluent enters a sand trap to separate the medium from the treated effluent to recirculate the medium in the reactor. Secondary clarification is not typically required. A portion of the fluidized-bed medium is pumped from the reactor to a sand-cleaning system, and the cleaned medium is recycled to the reactor. The washwater with solids content is concentrated and wasted with the primary sludge.

As for other biological attached-growth reactors, nitrification in the fluidized-bed reactor depends on the influent characteristics, including organic nitrogen and total ammonia-nitrogen loads, $cBOD_5$:TKN ratio, total ammonia-nitrogen concentration and oxygen availability. Fluidized-bed pilot studies have shown that volumetric removal rate and nitrification rate decrease with low ammonia concentrations. Nitrification activity tends to decrease with low temperature and will be sensitive to pH and alkalinity.

Mechanical scale-up and operation of fluidized-bed reactors have shown problems related to devices and methods for influent distribution, oxygen saturation and bed expansion control (WEF, 2000b). A draft-tube fluidized-bed reactor with a side-connected airlift to generate the required water velocity for media expansion by pumping and recirculation of the influent and the reactor content has been developed to address some of these issues.

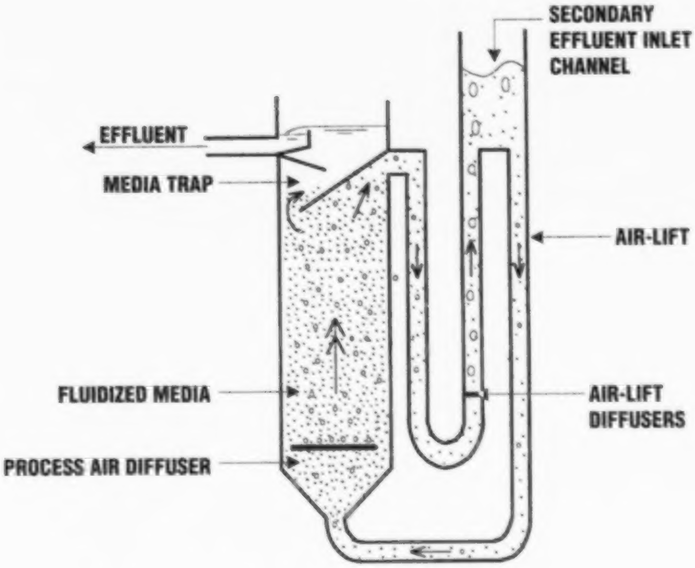
The Biolift process, manufactured by OTV, represents an application of this fluidization mode and has been applied at an operating scale of 2400 $m^3$/d (0.53 MIGD) at Maxeville, Nancy, France (WEF, 2000b). The full-scale reactor will be in operation in 2001 on the 300 000 inhabitant equivalent existing activated sludge plant of Maxeville (John Meunier Inc., technical communication, 2001).

The Biolift process has been developed for tertiary nitrification in an upflow fluidized-bed reactor with co-current process air diffusion. The process is continuously operating and does not require any backwash event. Figure 5.12 provides a schematic view of the reactor components. The Biolift reactor comprises the following:

- a side-connected airlift pumping the influent and the recirculated reactor content to the bottom of the Biolift reactor in an upflow mode;
- a process air diffuser network located at the bottom of the reactor;
- a media trap located on top of the reactor to reduce turbulence and media losses at the effluent; and
- an airlift intake located in the high portion of the reactor to develop recirculation.

The wastewater flow scheme through the Biolift reactor involves the pumping of the secondary-treated effluent by the airlift along with the recirculated reactor content at the base of the reactor to favour a high water velocity to control expansion of the medium and distribution of the influent. The diffuser network at the base of the reactor provided control of the process air diffusion required to maintain aerobic conditions and oxygen transfer in the fluid and the biofilm. The fluidized medium offers a high specific surface area for biofilm growth to enhance nitrification capacity. The media trap decreases the turbulence of reactor contents prior to effluent discharge and reduces media loss. The effluent enters a degas system to strip excess oxygen before the final clarification, as in the case of Maxeville discharge.

**Figure 5.12 Biolift Fluidized-Bed Reactor (OTV)**



The excess biomass control is achieved through airlift recirculation, providing turbulence of the recirculated reactor content. Thus, the excess biomass is sloughed off the medium. The sloughed solids are evacuated from the reactor with the effluent and settle in the final clarifier.

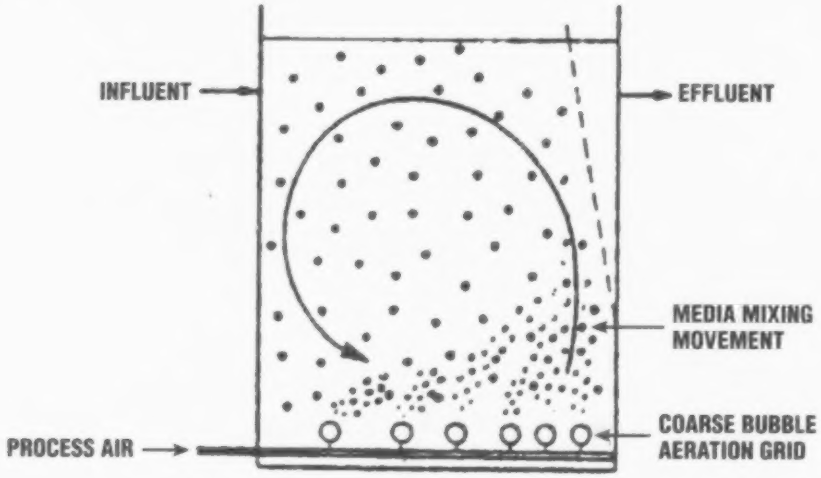
The Biolift process application has been applied only as tertiary nitrification following an existing activated sludge process. The Biolift process was inserted between the activated sludge process aeration tank and the final clarifier. The activated sludge process was converted to an anoxic reactor, and a Biolift effluent recirculation line was installed to recirculate the effluent nitrates to the anoxic zone.

#### 5.1.4.4.2 Moving-Bed Biofilm Reactor

The MBBR process has been developed by Kaldnes Miljoteknologi and consists of a continuously aerated operating system with a lightweight medium that does not require backwashing or recirculated sludge flows. The process has been tested and installed for industrial and municipal wastewater treatment (WEF, 2000b). Municipal applications are mostly located in northern Europe, and the process has been used for organic load removal, nitrification and total nitrogen removal.

The typical flow scheme for the MBBR process involves the primary treatment of the raw wastewater with screening and primary settling. The primary-treated effluent then enters the MBBR unit or train. The reactor is partially filled with medium, which supports a biofilm of attached-growth biomass. A coarse-bubble aeration system ensures the movement of the medium within the reactor and aerobic conditions, as shown in Figure 5.13. The lightweight rigid medium moves through the length of the reactor along with the wastewater. The biological film developed on the synthetic medium achieves the level of aerobic treatment

**Figure 5.13 Kaldnes MBBR Process, Relying on Lightweight Media Mixing Movement (WEF, 2000b)**



required to remove organic components and total ammonia-nitrogen. The reactor effluent enters a secondary clarifier or a flotation unit to separate the solids and the sloughed biomass. The clarified effluent is further treated or discharged.

The synthetic medium is kept within the process by the use of sieve assemblies located at the front and rear ends of each unit. The biomass thickness is controlled by the mixing turbulence in each process unit, which allows sloughing of the excess biomass on a continuous basis.

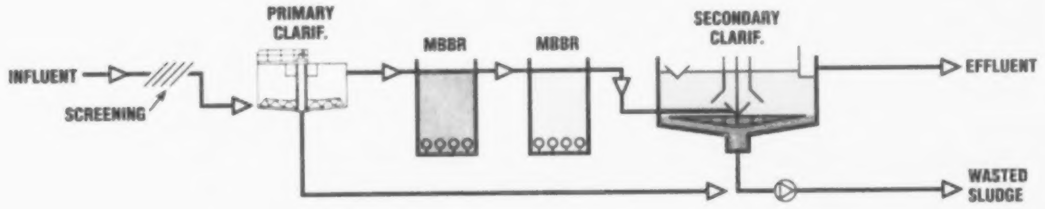
The MBBR process can be used for combined $BOD_5$ removal and nitrification, in a single stage. As illustrated in Figure 5.14, the MBBR units sequentially achieve the organic load removal in the first reactor and ammonia removal with nitrification in the second or following reactors. Tertiary nitrification, referred to as two-stage, involves a secondary-treated effluent with low $BOD_5$ content and high total ammonia-nitrogen. The secondary effluent may be generated by a previous biological treatment, such as conventional activated sludge. The two-stage system, with an intermediate clarification prior to the MBBR, permits high $BOD_5$ loading rates for organics removal in the first biological stage and generates a clarified low $BOD_5$ and high total ammonia-nitrogen effluent for tertiary nitrification in the MBBR.

The MBBR process may be applied as stand-alone reactors in configurations such as described above, or the medium could be added to existing activated sludge basins to increase the biomass content in the biological system.

### 5.1.4.5 Recirculating Sand Filter

Recirculating sand filters allow for secondary and tertiary wastewater treatment applications (Murphy *et al.*, 2000). The technology has been shown to achieve tertiary nitrification of small-flow lagoon effluent in continuous-discharge mode for the year-round control of total ammonia-nitrogen to below a concentration

**Figure 5.14 Kaldnes MBBR Single-Stage Process for Combined Organic Carbon Degradation and Nitrification (Kaldnes Miljøteknologi)**



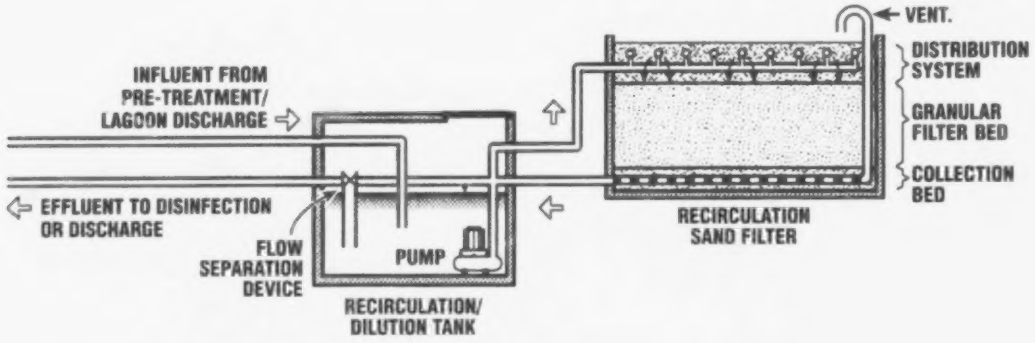
of 5 mg/L (Rushbrook and Urban, 1998; Olson and Danen, 2001).

Recirculating sand filters typically include a pretreatment unit followed by a recirculation/dilution tank and a granular medium filter bed. Figure 5.15 illustrates these units. The pretreatment unit could be utilized as the up-front physical pretreatment followed by the BOD, and suspended solids removal processes occurring in conventional facultative or aerated lagoons. The lagoon effluent enters the recirculation/dilution tank, where it is diluted and mixed with a recirculated portion of the filter bed-treated effluent. An automated dosing sequence periodically activates the recirculation pumping system, and the diluted wastewater is distributed onto the surface of the filter bed through a low-pressure distribution system. The wastewater percolates through the unsaturated filter bed medium where tertiary nitrification is achieved by a thin aerobic attached-growth biofilm on the medium. The effluent is collected in an underdrain system and conveyed to the recirculation/dilution tank. The treated effluent is typically high in DO. According to the liquid level in the recirculation/dilution tank, either a flow separator device returns all of the filter bed-treated effluent to the recirculation/dilution tank or a portion is evacuated to the disinfection unit/final discharge and the balance is returned to the recirculation/dilution tank. This mode of operation permits recirculation of the treated effluent.

The recirculation/dilution tank includes a set of pumps to feed and pressurize the filter bed distribution system and an automated control system for pump operation with on/off cycles to intermittently deliver wastewater to the filter bed. A flow separator mechanism is included as a float ball valve or a recirculating splitter valve that may completely or partially isolate the treated effluent outlet to the recirculation/dilution tank according to the liquid level in the tank. An external splitter basin can be provided to return a predetermined percentage of treated flow to the recirculation/dilution tank. Prefilters must be installed between the recirculation/dilution tank and the low-pressure distribution system to avoid plugging of the orifices.

The filter bed includes a low-pressure wastewater distribution system placed on top of the granular filter medium. The distribution system is generally polyvinyl chloride piping with calibrated orifices installed within a gravel/crushed stone layer. The filter medium is under the distribution system. The granular medium typically has an effective size of 1.0–5.0 mm (uniformity coefficient

**Figure 5.15 Recirculating Sand Filtration (Ministère de l'Environnement du Québec, 2001)**


VENT.
DISTRIBUTION SYSTEM
GRANULAR FILTER BED
COLLECTION BED
INFLUENT FROM PRE-TREATMENT/ LAGOON DISCHARGE
EFFLUENT TO DISINFECTION OR DISCHARGE
FLOW SEPARATION DEVICE
PUMP
RECIRCULATION/ DILUTION TANK
RECIRCULATION SAND FILTER

[UC] <2.5), must be non-degradable and has a varying depth of 0.6–0.9 m (Ministère de l'Environnement du Québec, 1992, 2001). A perforated collection drain is located at the bottom of the filter bed and covered with an underdrain granular layer. The underdrain granular material typically has an effective size of 10–20 mm and prevents filter medium from entering the collection drain (Metcalf & Eddy, 1991). The total filter surface required is usually divided into several filter bed sections, which permits isolation of at least one section of the filter beds for regeneration or maintenance. It is important to maintain aerobic conditions within the filter bed, as the development of anoxic and anaerobic conditions would reduce the efficiency of ammonia removal. Oxygenation of the recirculating sand filters may proceed by the following mechanisms:

- recirculation process permitting the introduction of oxygenated treated water and microorganism seed to the applied wastewater;
- intermittent wastewater application on each filter cell providing idle periods between contaminant load additions to favour aeration of the filter bed;
- collecting drains extending to the atmosphere at the surface of the filter bed to favour filter aeration; and
- vents located in the distribution system gravel layer/crushed stone extending to the surface of the filter bed to favour aeration.

### 5.1.4.6 Intermittent Sand Filter

Intermittent sand filters can allow secondary and tertiary wastewater treatment applications (USEPA, 1993). The technology has been shown to achieve tertiary nitrification of small-flow lagoon effluent with seasonal discharge for the control of total ammonia-nitrogen concentrations to below a monthly average concentration of 5 mg/L (McMaster University and WTI, 1996). Complete nitrification can be expected with the intermittent sand filter technology except under extremely cold conditions. Thus, intermittent sand filter operation for ammonia removal using small-flow lagoon effluent is restricted to warm weather or seasonal operation only (Murphy *et al.*, 2000), and lagoon treatment plants using intermittent sand filters must provide for winter storage in cold-climate applications.

The intermittent sand filter is a single-pass, gravity-driven, slow-rate filtration process, where the filter surface is flooded intermittently with lagoon effluent and allowed to drain (USEPA, 1992). The intermittent sand filter typically includes a pretreatment unit followed by a dosing tank and a granular medium filter bed. Figure 5.16 illustrates a section of an intermittent sand filter. As for the recirculating sand filter, the pretreatment unit could be utilized as the up-front physical pretreatment followed by lagoon treatment and storage pond. Thus, the lagoon effluent with low $BOD_5$ and high total ammonia-nitrogen enters the dosing tank. A dosing pump system intermittently applies wastewater on the surface of the filter bed through a low-pressure distribution system. The wastewater percolates through the unsaturated granular filter bed. A thin aerobic attached-growth biofilm on the granular medium achieves nitrification. The treated liquid is collected in an underdrain system located at the bottom of the filter and discharges to the disinfection unit, disposal field or surface waters. The intermittent sand filters do not include effluent recirculation through the filter bed.

The dosing tank includes a set of pumps to feed and pressurize the filter bed distribution system and an automated control system for intermittent pump operation. Prefilters must be installed between the dosing tank and the low-pressure distribution system to avoid plugging of the orifices. The filter bed includes a low-pressure wastewater distribution system located on top of the granular filter medium. The distribution system consists of a piping network with calibrated orifices installed in a layer of washed gravel/crushed stone. The filter medium under the distribution system consists of a non-degradable granular medium with a typical effective size of 0.25–1.0 mm (UC < 4) and a varying depth of 0.6–0.9 m (Metcalf & Eddy, 1991; Ministère de l'Environnement du Québec, 2001). The perforated underdrain is located at the bottom of the filter bed in a granular layer composed of washed gravel or crushed stone. Two different sizes of material are typically used for the underdrain. The first layer under the filter medium has an effective size of 2.5–5 mm, and the second layer, in which the underdrain is placed, has an effective size of 10–20 mm (Ministère de l'Environnement du Québec, 2001). This arrangement prevents the filter medium from migrating to the underdrain. The total filter surface is typically divided into a minimum of two or three parallel sections, which permits the isolation of at least one filter bed for regeneration or maintenance.

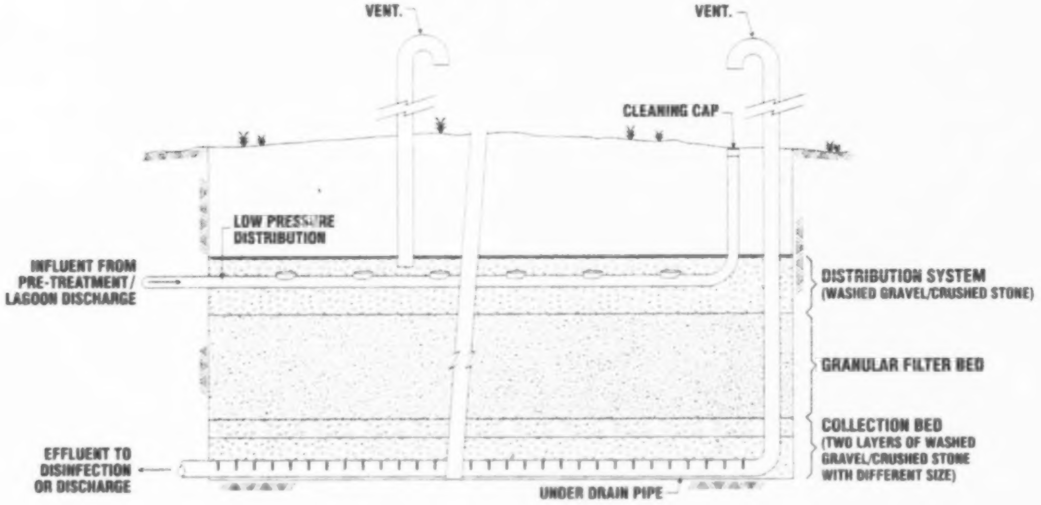
As for the recirculating sand filter, aerobic conditions within the filter bed are important to maintain nitrification efficiency. Oxygenation of the filter bed is supported with the following mechanisms:

- intermittent wastewater applications on each cell providing idle periods between contaminant load additions to favour the aeration of the filter bed;
- collecting drains extending to the atmosphere at the surface of the filter bed to favour filter aeration; and
- events located in the distribution system gravel layer/crushed stone extending to the surface of the filter bed to favour aeration.

### 5.1.4.7 Constructed Wetlands

Constructed wetlands are based on the wastewater treatment quality observed with natural marsh systems and have been identified as achieving ammonia removal (NovaTec Consultants Inc., 1996). The treatment performance can be duplicated and improved with specific wetland construction considerations. The technology has been shown to allow secondary and tertiary wastewater treatment and represents an upgrade

**Figure 5.16 Intermittent Sand Filtration (Ministère de l'Environnement du Québec, 2001)**


VENT.
VENT.
CLEANING CAP
LOW PRESSURE DISTRIBUTION
INFLUENT FROM PRE-TREATMENT/ LAGOON DISCHARGE
DISTRIBUTION SYSTEM (WASHED GRAVEL/CRUSHED STONE)
GRANULAR FILTER BED
COLLECTION BED (TWO LAYERS OF WASHED GRAVEL/CRUSHED STONE WITH DIFFERENT SIZE)
EFFLUENT TO DISINFECTION OR DISCHARGE
UNDER DRAIN PIPE

option for tertiary nitrification of small-flow lagoon effluents with continuous or seasonal discharge. Constructed wetlands are also climate dependent.

Constructed wetlands can be installed where land characteristics and climate conditions are favourable. The selection of the site for a wetland location depends on site-specific factors, including the following:

- site proximity to existing lagoon to avoid pumping requirements and maintain final discharge location;
- low-relief area with minimal shallow bedrock to reduce site preparation activities;
- uniformity and permeability of soils and bedrock characteristics, as highly permeable soils and limestone bedrocks may require liners;
- seasonal groundwater table depth to avoid interference with wetland water content; and
- flooding potential, which may require protection structures, including elevated dykes.

Constructed wetlands under cold weather are subject to biological activity reduction and ice cover formation, which may lead to flow restriction. Storage of wastewater in lagoons during the winter months and discharge to a seasonal wetland may be considered to ensure satisfactory total ammonia-nitrogen removal.

Constructed wetlands are classified into two types — namely, free water surface (FWS) wetlands and vegetated submerged beds (VSBs), previously referred to as subsurface flow wetlands. FWS wetlands are close to natural wetlands in their appearance, as they contain floating and emergent aquatic plants rooted in the soil layer on the bank of the wetland with wastewater circulating between the leaves and stems of the plants. An open aerated zone located in the centre of the wetland contains some submerged-growth plants. A schematic view of an FWS wetland is shown in Figure 5.17. Lagoon effluent is typically discharged to the wetland inlet system to equally distribute the hydraulic and total ammonia-nitrogen load on the width of the system. The wastewater flows from the bank of the wetland (zone 1), where aquatic plants are growing, to an open-water area (zone 2), followed by the opposite bank (zone 3), where the outlet structure is located. The surface water is exposed to the atmosphere on all the surface area of the wetland. According to the soil and groundwater characteristics, a liner may be installed at the bottom of the wetland under the soil layer to contain wastewater and avoid groundwater contamination. FWS wetlands are usually installed with a minimum of two parallel cells.

The FWS wetland banks typically have an average water depth of 0.3 m, but depth may vary from 0.15 to 0.75 m. The open-water zone represents approximately 20% of the FWS wetland surface area, with a minimum water depth of 1.2 m to prevent colonization by emerging plants. The open-water zone allows for flow redistribution and atmospheric surface aeration. The typical length:width ratio ranges from 1:1 to 4:1 (NovaTec Consultants Inc., 1996; USEPA, 2000).

VSBs do not resemble natural wetlands, as the water surface is located below the top level of the medium, and they generally do not support aquatic wildlife. Thus, there is no surface water exposed to the atmosphere, and the surface water is referred to as subsurface. The VSB basin contains a media bed of either coarse sand or medium-sized gravel, used as aquatic plant root support through which the subsurface wastewater circulates. A schematic view of a VSB is shown in Figure 5.18. Lagoon effluent is discharged to the wetland inlet system to equally distribute the hydraulic and total ammonia-nitrogen load on the width of the VSB. The wastewater flows beneath the surface of the medium from the inlet zone of the wetland through the treatment zone followed by the outlet zone, where the discharge structure is located. The treatment zone medium is populated with the aquatic plants and their penetrating roots where the subsurface wastewater flows in contact with the root and rhizome system of the plants. As for the FWS wetland, according to the soil and groundwater characteristics, a liner may be installed at the bottom under the soil layer to contain wastewater and avoid groundwater contamination. The VSBs are usually installed with a minimum of two cells in parallel.

The VSB media bed in the treatment zone typically has an average depth slightly greater than 0.6 m, including a top fine gravel layer of 0.1–0.15 m. The water depth may vary from 0.3 to 0.6 m. The medium gravel size range is reported as 20–30 mm. The typical length:width ratio ranges from 1:1 to 4:1 (NovaTec Consultants Inc., 1996; USEPA, 2000).

Emergent aquatic plants are a necessity in both FWS and VSB constructed wetlands; however, the ammonia uptake by the vegetation represents a rather small portion of the treatment. The majority of the total ammonia-nitrogen uptake is achieved mainly by the attached-growth microorganisms on the submerged parts of the plants, on the litter for the FWS wetland and by the attached microorganisms on the plant roots and on the medium surface for the VSB. Since the gravel medium with the VSB has more surface area than the FWS wetland, the VSB may have higher reaction rates, which results in smaller wetland area compared with the FWS wetland (NovaTec Consultants Inc., 1996).

Aeration modes to sustain oxygen transfer in FWS wetlands include atmospheric surface aeration in the open-water zone and in the bank

**Figure 5.17 Free Water Surface Wetlands (USEPA, 2000)**

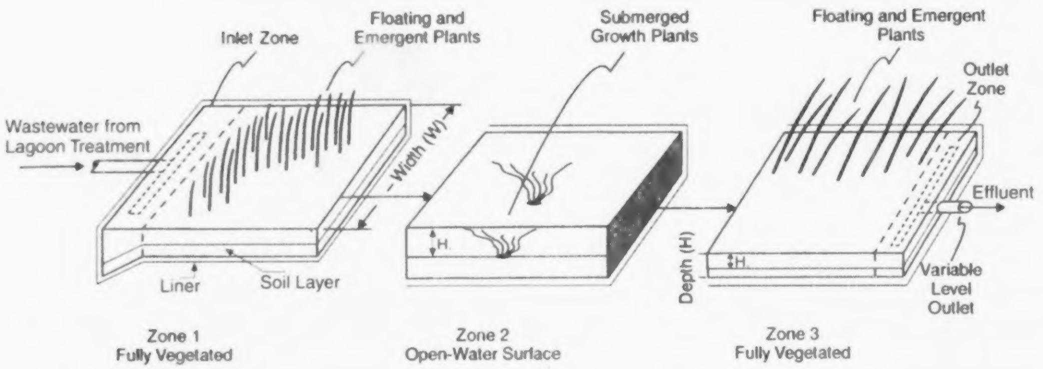
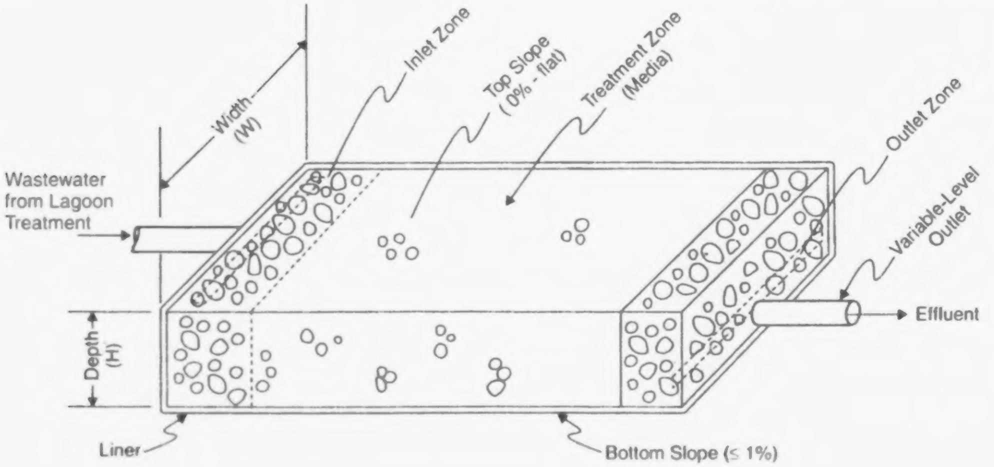

Inlet Zone
Floating and Emergent Plants
Submerged Growth Plants
Floating and Emergent Plants
Outlet Zone
Wastewater from Lagoon Treatment
Width (W)
Effluent
H.
Depth (H)
H.
Liner
Soil Layer
Variable Level Outlet
Zone 1 Fully Vegetated
Zone 2 Open-Water Surface
Zone 3 Fully Vegetated

**Figure 5.18 Vegetated Submerged Beds (USEPA, 2000)**


Width (W)
Inlet Zone
Top Slope (0% - flat)
Treatment Zone (Media)
Outlet Zone
Variable-Level Outlet
Wastewater from Lagoon Treatment
Depth (H)
Effluent
Liner
Bottom Slope (≤ 1%)

zones, due to shallow depth, and photosynthesis by the phytoplankton and the submerged plants.

Aeration capacity of the VSB is reported to occur through limited atmospheric oxygen transfer and, to some extent, through the plant stems and their penetrating roots. As the water level is maintained below the media bed surface, atmospheric aeration rate may be significantly less than for FWS wetlands (NovaTec Consultants Inc., 1996). There is no clear consensus on oxygenation efficiency of the plant stems and roots. It has been reported that roots and rhizomes of the aquatic vegetation have surface aerobic sites that are repeatedly placed in contact with the circulating wastewater through the depth of the roots' penetration in the media bed (NovaTec Consultants Inc., 1996). Thus, the root system of the chosen aquatic plants must ideally penetrate the full depth of the media bed to enhance the relative percentage of aerobic sites. The media bed zone located under the roots of the aquatic plants may suffer from low aerobic sites and present a rather anaerobic environment. There are indications that the rate of nitrification might be related to the root penetration depth zone (NovaTec Consultants Inc., 1996). Alternating flow in parallel VSBs on a frequent basis favours atmospheric reaeration of the media bed and deeper root penetration.

A variety of aquatic plants have been used for FWS and VSB wetlands. The type of vegetation associated with the VSB should be related to the root penetration depth potential. The emergent aquatic plants typically found with constructed wetlands for wastewater treatment include the following (NovaTec Consultants Inc., 1996):

- cattail, shallow root penetration at 0.3 m, common use on several FWS wetlands and VSBs, but less desirable for VSBs due to the shallow root penetration, unless the media bed depth is adjusted;
- bulrush, deep root penetration at 0.6 m, common use on VSBs; and
- reeds, deep root penetration at 0.4 m, successful utilization on constructed wetlands for wastewater treatment, potential use to avoid muskrat damage because of its low food value.

Other aquatic plants for potential use include phragmites, sedges and rushes. The phragmites have been successfully used for wetland construction in Europe and for some applications in Canada.

### *5.1.5 Combined Suspended-Growth and Attached-Growth Nitrification Processes*

The combination of suspended-growth and attached-growth systems for nitrification applications can involve either dual biological processes or hybrid processes, as described below.

#### 5.1.5.1 Dual Biological Processes

Dual biological treatment for total ammonia-nitrogen control involves the combination of an attached-growth aerobic process stage (e.g., trickling filter) for cBOD removal in series with a downstream suspended-growth nitrification stage. Various process combinations are possible, such as the activated biofilter, biofilter/activated sludge, trickling filter/solids contact and series trickling filter/activated sludge processes. These process combinations and the individual parent processes are described in detail in WEF and ASCE (1992, 1998). For enhanced performance of the downstream nitrification stage, the trickling filter/solids contact process includes provision for reaeration of the RAS stream, while the trickling filter/activated sludge process could utilize an intermediate clarifier.

When properly configured, dual biological processes can combine the advantages of attached-growth and suspended-growth treatment technologies, some of which were described in Sections 5.1.3 and 5.1.4, while minimizing their respective disadvantages. Experiences with dual biological processes for achieving complete nitrification (e.g., with a combination roughing trickling filter and activated sludge process) are described in USEPA (1989a) and Daigger and Buttz (1998).

#### 5.1.5.2 Hybrid Biological Processes

Hybrid biological treatment processes comprise both suspended-growth and attached-growth biomass in a single-stage aerobic process, which can offer technical advantages. The combination of fixed-film and suspended-growth biomass can result in relatively high levels of active biomass and SRT, allowing for enhanced treatment capacity and nitrification and increased process stability. Hybrid systems are typically implemented as retrofits of existing aerobic biological treatment plants and represent a potentially cost-effective upgrade option for achieving enhanced nitrification.

A typical hybrid design is the integrated fixed-film activated sludge (IFAS) process, which involves the addition and retention of attached-growth support media within the aeration basin. There are a number of successful full-scale applications of this technology. A schematic example of an IFAS process could include a modification of the MBBR process configuration shown in Figure 5.14, but where an RAS stream from the secondary clarifier to the aeration tank is included. Various types of media can be used as biomass growth support surfaces, as described below.

The IFAS process relies partly on acclimated mixed liquor provided by the RAS stream from the secondary clarifier. However, since mixed liquor suspended solids (MLSS) concentration can be lowered as part of upgrading to an IFAS process, there are no negative impacts on the solids loading rate to the existing secondary clarification stage. The treatment process is less reliant on the settling characteristics of the suspended sludge in the final clarifier, since a significant portion of the active biomass is attached to the medium and retained within the aeration tank. Other beneficial characteristics of the IFAS process are a lower critical sludge age of the suspended nitrifying biomass, increased sludge settleability and reduced overall sludge yields.

Another possible hybrid process application could involve the retrofitting of existing wastewater treatment lagoons. Attached-growth support media could conceivably be added into one or more of the lagoon cells or into a segregated zone(s) within the lagoon process. Such a retrofit may require the installation of a diffused aeration system into the zone containing the support medium to ensure proper mixing conditions and the maintenance of a DO residual of greater than 3 mg/L. An adequate amount of medium is required to ensure year-round nitrification to target discharge levels. Although there are no reported full-scale examples of this type of lagoon process upgrade, the concept has been evaluated at the pilot scale in the United States (personal communication, May 2000).

A wide variety of attached-growth support media are commercially available for use in hybrid systems and have had varying degrees of success. The various types of media include:

- rope (i.e., attached to frames, placed in the aeration basin; also referred to as looped-cord media);
- fabric (i.e., textile pads attached to frames, placed in the aeration basin);
- sponge (i.e., small free-moving porous polyurethane foam cubes);
- plastic carrier (i.e., small, free-moving, rigid plastic carrier elements with high specific surface area; typical designs have high voidage and internal protected surface areas, while some media are solid particles, sometimes incorporating immobilized nitrifying microorganisms);
- fixed plastic (i.e., rigid plastic, immobile modular design with low to medium specific surface area); and
- powdered activated carbon (i.e., becomes incorporated into activated sludge flocs).

A summary of alternative attached-growth support media along with respective vendors and contact information is given in Appendix A. Additional details on hybrid systems, including media characteristics, as well as process design, operational and economic aspects and full-scale experiences, can be found in WERF (2000).

### 5.1.6 Other Biological Processes for Ammonia Removal

#### 5.1.6.1 Facultative Lagoons

Ponds in which wastewater stabilization is carried out by the combination of aerobic, anaerobic and facultative bacteria are known as facultative lagoons. Treatment is achieved in three distinct zones — namely, the surface zone, where aerobic degradation occurs, the bottom zone, where sediment is decomposed by anaerobic bacteria, and an intermediate zone that is partly aerobic and partly anaerobic, in which the decomposition of organics is performed by facultative bacteria (Metcalf & Eddy, 1991).

Conventional facultative lagoons are economical low-rate suspended-growth processes that can be designed as continuous-discharge or as fill-and-draw systems for seasonal or annual discharge. The requirement for storage capacity allowing for seasonal- or annual-discharge operations usually depends on site-specific climatic conditions and receiving water characteristics and constraints.

Facultative lagoons are typically configured as multicell systems that are larger and shallower than aerated lagoons and do not employ mechanical aeration or mixing. They may comprise anaerobic pretreatment cells for sedimentation and fermentative pretreatment, followed by the facultative treatment cells.

Facultative lagoons can offer effective treatment under ideal conditions, producing an effluent quality similar to that achieved by conventional mechanical secondary treatment processes. However, the low-rate process is susceptible to low wintertime temperatures as experienced in cold-climate locations such as Canada. Consequently, ammonia removal can become severely limited during winter periods due to reduced biological activity, with elevated effluent total ammonia-nitrogen concentrations (i.e., 10–25 mg/L) resulting during winter and spring operations for continuous-discharge plants and during the springtime for seasonal-discharge lagoons in Canada (Alberta Environment, 1987; OMOE and Environment Canada, 1993).

To achieve target effluent ammonia-nitrogen discharge concentrations of less than 5 mg/L for cold-climate continuous-discharge facultative lagoon applications in Canada, add-on tertiary nitrification would be necessary. Alternatively, storage cells are often employed, allowing for wastewater accumulation and effective long-term treatment of the accumulated wastewater during the summertime and the discharge of well-treated effluent in autumn. Effluent discharge characteristics from annual-discharge lagoon systems in Canada include total ammonia-nitrogen concentrations of less than 5 mg/L (Alberta Environment, 1987; OMOE and Environment Canada, 1993).

Facultative lagoon systems rely on the growth and photosynthetic activity of algae in the summertime and atmospheric reaeration for the supply of oxygen for biodegradation of organic matter. Ammonia removal can be significant during algae-controlled operations, relying on warm conditions and long daylight periods. Some of the removal is achieved through gaseous ammonia stripping to the atmosphere, which is enhanced by substantial increases in pH (i.e., to pH 8.5 or higher) due to carbon dioxide consumption through photosynthesis in the summertime. Total ammonia-nitrogen is also removed through assimilation in microbial and algal biomass and through a modest degree of ammonia oxidation, followed by nitrate removal by denitrification, depending on process conditions (USEPA, 1983, 1992, 1993; Daigger and Buttz, 1998; WEF, 1998). Harvesting of algal biomass is an option for maximizing the removal of ammonia through assimilation into biomass.

Facultative lagoon processes are in widespread use in Canada, particularly in the Prairie provinces, the territories and southwestern Ontario. The majority of conventional facultative lagoon systems employed in Alberta include a 12-month storage lagoon for annual release of high-quality effluent (Alberta Environment, 1987; Environment Canada, 1987).

## 5.2 Physical/Chemical Processes for Ammonia Removal

Physical/chemical removal of ammonia can involve conversion into nitrogen gas, stripping of un-ionized ammonia, ion exchange or membrane separation. This can be accomplished by the following technologies:

- breakpoint chlorination;
- air stripping;
- ion exchange; and
- membrane separation (e.g., reverse osmosis).

Although there have been full-scale applications of these ammonia removal processes, they have essentially been replaced by more conventional and feasible biological process technologies (WEF, 1998). It was emphasized in USEPA (1993) that physical/chemical treatment for removal of ammonia, except in highly specialized situations, is the process of last resort, especially at small plants.

Physical/chemical processes are considered to be applicable mainly as polishing stages following biological treatment processes when the degree of ammonia removal is insufficient to meet discharge targets. For example, polishing could be applied on a short-term basis for the removal of residual total ammonia-nitrogen from aerated lagoon process effluent during cold-climate operations, to compensate for reduced treatment capabilities of the biological process under these conditions. Ion exchange and breakpoint chlorination may be technically feasible options for supplemental treatment, as they are much less susceptible to low-temperature effects than are biological processes.

### 5.2.1 Breakpoint Chlorination

Breakpoint chlorination follows a series of complex reactions, including the oxidation of ammonium-nitrogen to nitrogen gas. About 90% removal of ammonium-nitrogen is achieved once the breakpoint reactions occur, which is represented stoichiometrically as:

$$2NH_4^+ + 3Cl_2 \rightarrow N_2(g) + 6Cl^- + 8H^+$$

Because some of the ammonia is converted into nitrate and other oxidized nitrogen compounds, the actual practical chlorine dose is often approximately 10 mg/mg ammonium-nitrogen (WEF, 1998) or higher (USEPA, 1993). Dechlorination must be considered to minimize the potential for aquatic toxicity due to residual chlorine.

Application of the breakpoint chlorination process can include the polishing of effluents from continuous-discharge aerated lagoons during seasonal low temperature conditions, where the biological process suffers from reduced ammonia removal efficiency. An example of such an application is at the lagoon-based water pollution control plant in Niagara-on-the-Lake, Ontario, which employs breakpoint chlorination for short periods during the wintertime.

### 5.2.2 Ion Exchange

Ion exchange is an ion-specific technology that may be used for selective removal of ammonium-nitrogen. Organic nitrogen in raw wastewater may need to be converted to ammonium-nitrogen prior to ion exchange treatment to ensure a consistent low level of effluent total ammonia-nitrogen. Alternatively, the process could conceivably be considered for posttreatment of secondary-treated effluent (e.g., from lagoon processes), but needs to be preceded by tertiary filtration for the removal of suspended solids. The process is best suited for applications experiencing prolonged periods of freezing conditions, yet where ammonia removal must be consistently maintained (USEPA, 1971). However, ion exchange technology is expensive.

The ion exchange technology consists of feeding filtered wastewater to a packed column or bed of specific natural or synthetic zeolite resin. The wastewater passing through the column exchanges one ion for another ion from the zeolite resin. The column is operated until exchange sites on the zeolite resin are exhausted, which is revealed by high effluent residual ammonium, referred to as breakthrough. The resin is periodically regenerated by displacing the removed ions from the exchange sites by passing a regenerant brine solution through the column or bed. The regenerant can then be further processed for recovery and reuse or wasted using appropriate disposal options (USEPA, 1971, 1993).

### 5.2.3 Air Stripping

Air stripping technology appears best suited for polishing of lagoon effluents during warm-temperature operations, if necessary, since performance is highly temperature dependent. Air stripping tower operations are effectively limited to wastewater temperatures of greater than 10°C to avoid freezing and associated operational problems (USEPA, 1993). Applicability for the polishing of lagoon effluents during cold-weather operations in Canada is, therefore, limited.

### 5.2.4 Reverse Osmosis

Reverse osmosis is a non-ion-specific technology using a high-pressure membrane capable of generating ultra-high-purity treated effluent. The reverse osmosis process consists of feeding wastewater through a high-pressure pump in a module system including a pressurized vessel and a semipermeable membrane. The pressure applied on the wastewater in the vessel is greater than the natural osmotic pressure that normally allows pure water diffusion through a semipermeable membrane towards a concentrated solution to equalize solution strength. The applied pressure in excess of osmotic pressure reverses the natural water flow direction from concentrated solution to dilute solution, which explains the term reverse osmosis. The semipermeable membranes used in reverse osmosis are not permeable to dissolved salts, inorganic molecules and organic molecules with molecular weights greater than 100 (WEF and ASCE, 1998). The pressurized wastewater is separated into a low-saline portion going through the membrane, referred to as the permeate, and a high-saline portion retained by the membrane, referred to as the concentrate.

Application of reverse osmosis typically includes desalination, food and beverages processing, and industrial wastewater reuse and products recovery. Reverse osmosis can be used to remove nitrogen compounds with other dissolved constituents from wastewater. Application of reverse osmosis in two locations in the United States (Orange County, California, and Denver, Colorado) has shown ammonium-nitrogen removals of 94% and 85%, respectively (USEPA, 1993). The total ammonia-nitrogen residuals in the Orange

County application were lower than 2 mg/L. Nitrates were separated in the same application, but with a significantly lower removal percentage of 55%.

## 5.3 Enhanced Nutrient Removal

### *5.3.1 Biological Denitrification*

Denitrification allows for total nitrogen removal. It is a multistep process that involves the reduction of nitrates formed during nitrification of municipal wastewaters to molecular nitrogen gas. The denitrification process is achieved by heterotrophic facultative microorganisms that utilize nitrates instead of oxygen in their energy production pathway. Thus, an anoxic environment, characterized by the absence of oxygen and the presence of nitrates, is required to permit denitrification. If both oxygen and nitrates are present, microorganisms preferably utilize oxygen first and then proceed with nitrates in the denitrification process. In cases where ammonium-nitrogen is not sufficient as a nutrient for the cell synthesis needs, some microorganisms may reduce nitrate to ammonium for this use. The latter is referred to as assimilatory nitrate reduction, as it is incorporated in the cell, to distinguish it from the denitrification process.

A carbon substrate source is required for the denitrifying microorganisms to achieve the denitrification process and fulfil cell synthesis requirements. The carbon source may come from the primary treated wastewater, the endogenous decay of biomass or various external organic compounds, such as methanol, ethanol, acetic acid, acetone or other industrial organic wastes (USEPA, 1993). Further information on denitrification influencing factors and process rates can be found in Chapter 4.

MWWTPs for ammonia removal may be upgraded with denitrification to incorporate benefits related to the latter process. This section briefly presents the advantages associated with denitrification and the process configurations that promote denitrification process benefits in ammonia removal MWWTPs.

#### 5.3.1.1 Denitrification Process Benefits

Process benefits from denitrification activity may have significant impacts on the sludge characteristics and the aerobic processes. The denitrification benefits include the following:

- **Sludge settleability characteristics**. Denitrification of a mixed liquor with a high nitrate content prior to clarification significantly reduces or avoids rising sludge problems. Rising sludge is caused by denitrification within the settled sludge (below the sludge blanket) where nitrogen gas formed is trapped in the sludge mass, which may become buoyant and rise or float to the surface (Metcalf & Eddy, 1991). Anoxic cells located at the head end of a biological treatment train may be used as a biological "selector" to reduce the growth of filamentous bacteria and promote zoogleal organisms, which enhance biomass flocculation quality (USEPA, 1993).
- **Alkalinity production**. Alkalinity is produced by 3.57 g of calcium carbonate per gram of nitrate reduced if nitrate is the nitrogen source for cell synthesis (assimilatory nitrate). Alkalinity production may range from 2.9 to 3.0 g of calcium carbonate per gram of nitrate reduced if ammonium-nitrogen is available. Thus, chemical requirements for pH adjustment in low-alkalinity wastewaters, which are sensitive to pH depression from the nitrification process, are reduced or eliminated (USEPA, 1993; WEF, 1998).
- **Carbon removal and reduced oxygen requirement**. Consumption of primary treated wastewater carbon ($BOD_5$) by heterotrophic microorganisms during the denitrification process reduces the $BOD_5$ load applied in the

subsequent aerobic zone (USEPA, 1993). Thus, the oxygen requirement is proportionally reduced. The oxygen demand is reduced by 2.86 g of oxygen per gram of nitrate reduced (WEF, 1998). The reduction of $BOD_5$ load reduces the $cBOD_5$:TKN ratio and increases nitrification potential in the aerobic zone.

### 5.3.1.2 Denitrification Process Configurations

Denitrification process configurations may be divided into two basic categories — namely, single sludge and separate stage. Single sludge implies nitrification and denitrification processes within the same biological treatment train. Separate stage achieves denitrification by treating the effluent from a nitrifying process in a distinct biological train operated independently from the carbon removal nitrification train.

Single-sludge systems involve a series of anoxic and aerobic cells or cycles followed by a final clarifier. Fewer tanks and clarifiers are required when nitrification and denitrification are combined. Single-sludge systems have been used to a much greater extent for municipal wastewater (WEF, 1998). Different configurations have been used and developed with single-sludge systems, including predenitrification, postdenitrification and combined pre- and postdenitrification.

Separate-stage processes involve two different trains of cells with dedicated clarifiers to each train. The first train is to ensure carbon removal and nitrification, and the second train is to achieve denitrification of the first-train effluent.

Single-sludge systems offer a potential cost advantage over the separate-stage process, as no intermediate clarifiers or separate denitrification units are involved and less space is required. Single-sludge systems can more readily be used in retrofitting existing activated sludge plants for total nitrogen removal, especially when the plant has excess capacity (WEF, 1998). However, single-sludge systems may be more susceptible to toxicity or inhibition owing to the absence of a separate upstream biological treatment step. Separate-stage processes offer the advantage of a separately optimized denitrification process to attain low total nitrogen concentrations, but typically requires an alternative carbon source such as methanol or industrial organic carbon wastes. Higher O&M costs are also associated with separate-stage processes.

Among the single-sludge configurations, predenitrification is the configuration that provides the denitrification advantages without requiring an external carbon source other than raw primary-treated wastewater. Thus, single-sludge predenitrification configurations are mainly discussed in this section, and brief references are made to postdenitrification and combined pre- and postdenitrification configurations. Separate-stage configurations may involve either suspended-growth or attached-growth processes, and more information may be found in several references, such as Metcalf & Eddy (1991), USEPA (1993), WEF (1998, 2000b) and WEF and ASCE (1998).

#### 5.3.1.2.1 Predenitrification Configurations

Predenitrification mode of operation involves an anoxic zone preceding the aerobic zone. Carbonaceous and nitrogenous demand of the wastewater is met in the aerobic zone. Following nitrate formation, the mixed liquor is recycled back from the aerobic zone to the front-end anoxic zone to allow denitrification. Primary-treated raw wastewater is typically used as the carbon source for the denitrification process in this anoxic zone. The predenitrification zone may serve as a buffer zone against shock loads to the sensitive nitrifier organisms in the aerobic zone.

The size of the anoxic zone depends on the amount of nitrogen to be removed, but typically the anoxic zone represents approximately 20–40% of the total volume (i.e., anoxic and aerobic) (WEF, 1998). The biomass in the anoxic zone must be kept in suspension without entraining DO, which inhibits the process. Minimum DO should be incorporated in the anoxic zone from the influent and the internal recycle nitrified mixed liquor. Suspended-growth predenitrification systems typically have 6–30 days SRT (with SRT of the system increased in proportion to the volumetric fraction of the anoxic zone), 100–400% of the design flow for the nitrified mixed liquor recycle and 30–100% of the design flow for RAS (USEPA, 1993; WEF, 1998). In terms of denitrification efficiency, single-sludge systems with one anoxic zone may reduce total nitrogen to 4–11 mg/L (WEF, 1998).

Predenitrification configurations include systems such as the Modified Ludzack-Ettinger, the anaerobic/anoxic/oxic ($A^2/O$) process, multi-anoxic zone step feed, oxidation ditches, SBRs, aerated submerged fixed-bed reactors (BAF) and MBBR.

The Modified Ludzack-Ettinger process involves an anoxic denitrification zone ahead of the aerobic zone using raw primary wastewater as the carbon source. The nitrified mixed liquor is internally recycled from the aerobic zone to the anoxic zone. RAS is recycled from the clarifier to the anoxic zone. This configuration ensures adequate nitrates for the heterotrophic denitrification population. Figure 5.19 provides a schematic of the Modified Ludzack-Ettinger configuration.

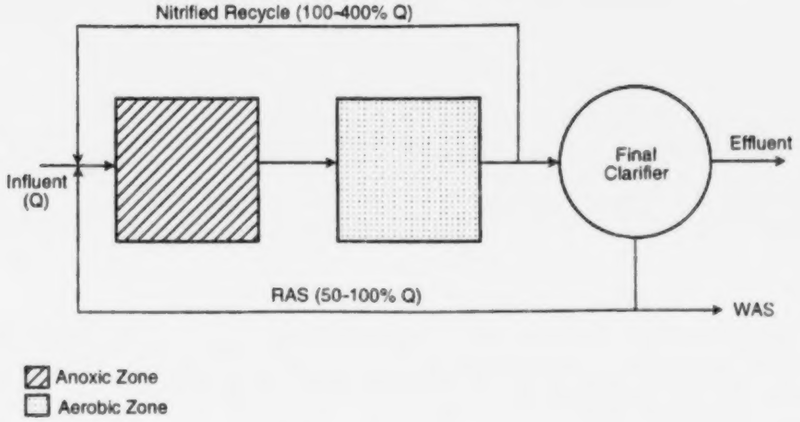
The $A^2/O$ process was originally developed for EBPR as the anaerobic/oxic process. Nitrification-denitrification was incorporated with the addition of the anoxic zone between the anaerobic and aerobic zone. The anaerobic zone may be used at the start of the train as an anaerobic selector for applications that do not require EBPR. Figure 5.20 provides a schematic of the $A^2/O$ configuration. Both the Modified Ludzack-Ettinger and $A^2/O$ processes can be optimized with the internal nitrified recycle and the RAS rates.

Multi-anoxic zone step feed is based on a series of coupled anoxic/aerobic zones, typically three stages in series of coupled anoxic/aerobic zones, with step feed of raw primary wastewater in each anoxic zone. The staging of the aerobic/anoxic zones replaces the use of an internal nitrified mixed liquor recycle. Each anoxic zone is sized to denitrify the nitrates produced in the preceding aerobic zone.

Oxidation ditches may achieve predenitrification with different configurations. The Orbal Sim-Pre process is an acronym for simultaneous nitrification-denitrification/predenitrification. The Orbal Sim-Pre process incorporates a first aeration channel with raw primary influent. Since the aeration demand exceeds the air supply, anoxic conditions are attained along the flow path in the area upstream of the aerators. Thus, simultaneous nitrification-denitrification occurs in the first aeration channel. The following aeration channels are aerobic zones where complete nitrification is achieved. An internal recycle of the nitrified mixed liquor from the innermost to the outermost channel ensures the predenitrification process. Other oxidation ditch configurations include two-ditch and three-ditch types, such as the BioDenitro DE and BioDenitro T processes, respectively. The latter configurations achieve predenitrification and aerobic conditions alternately within each looped reactor (Appendix A).

SBRs operate in a discontinuous fill-and-draw mode with the process biomass cycling through

**Figure 5.19 Modified Ludzack-Ettinger Process Schematic (WEF, 1998)**



anoxic and aerobic conditions through the cycle. A typical SBR cycle including predenitrification involves a batch feed period followed by mix-fill and react-mix periods without aeration for predenitrification of the remaining nitrates following the previous decant period. A react period with aeration follows to fulfil carbonaceous and nitrogenous oxygen demand, and the cycle ends with settle and decant periods.

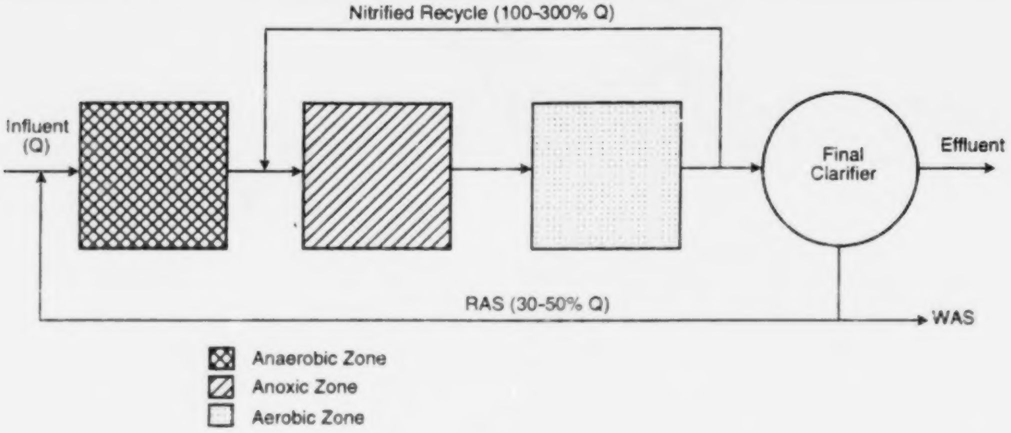
Attached-growth systems may be applied in predenitrification configurations. Aerated submerged fixed-bed reactors (BAF) include two types of reactors that may achieve predenitrification. The Biofor process may be used as a complete separate unit located at the front end of a biological treatment process that receives raw primary-treated wastewater and recycled nitrified effluent from the downstream biological treatment. The Biofor unit is operated as a complete anoxic zone without aeration. The Biostyr process may achieve predenitrification, carbon removal and nitrification within the same unit, as a second air header network could be located within the medium depth to favour a predenitrification anoxic zone under the air header (Section 5.1.4). The anoxic zone of the Biostyr process receives the primary-treated wastewater and the recycled nitrified effluent from the Biostyr unit.

The MBBR process may include an upstream anoxic zone including submersible mixers to maintain the medium in suspension without aeration. As for activated sludge, predenitrification is achieved by recycling the nitrified mixed liquor from the aerobic MBBR into the anoxic zone, and the carbon source is provided by the primary-treated effluent.

#### 5.3.1.2.2 Postdenitrification and Combined Pre- and Postdenitrification

Postdenitrification is achieved in an anoxic zone located downstream of the aerobic zone where aerobic carbon removal and nitrification are performed. By the time the wastewater reaches the anoxic zone, the carbon source is depleted, and denitrification occurs by endogenous decay. An external carbon source such as raw primary wastewater, methanol or industrial organic wastes

**Figure 5.20 A²/O Process Schematic (WEF, 1998)**



can be added to stimulate denitrification. The major disadvantage comes from a low COD:N ratio, which reduces the rate of denitrification and requires a larger anoxic volume compared with predenitrification. Postdenitrification should include a short aeration period prior to the final clarifier to ensure a minimum DO level prior to final clarification. Postdenitrification could be achieved with suspended-growth and attached-growth configurations, and more information may be found in references such as WEF (1998) and WEF and ASCE (1998).

Combined pre- and postdenitrification is typically required in cases where total nitrogen must be reduced to 1–3 mg/L (WEF, 1998). In such a case, pre- and postdenitrification configurations may be used to meet strict total nitrogen effluent requirements. The use of two anoxic zones is depicted with the four-stage Bardenpho process, which first includes one anoxic and one aerobic zone, which operates as predenitrification with internal nitrified mixed liquor recycle to the anoxic zone. The third and fourth zones are respectively anoxic and aerobic zones, which ensure final denitrification and a positive DO level prior to clarification. Other configurations and processes, such as the modified University of Cape Town process, SBR, MBBR and others, can be used, and more information may be found in references such as Metcalf & Eddy (1991), USEPA (1993), WEF (1998) and WEF and ASCE (1998).

### *5.3.2 Phosphorus Removal*

Phosphorus removal in MWWTPs may be achieved with either EBPR or chemical phosphorus removal. This section summarizes the major aspects of EBPR and chemical phosphorus removal with consideration given to ammonia removal.

#### 5.3.2.1 Enhanced Biological Phosphorus Removal

EBPR is based on phosphate accumulation by the activated sludge biomass. Phosphorus in excess of cellular growth normal requirement is stored by the biomass under specific alternating anaerobic and aerobic conditions.

The EBPR process is initiated in a front-end anaerobic zone where the readily biodegradable COD substrate is stored within the phosphorus-storing microorganism cells in the form of organic polymers. The intracellular storage of the readily biodegradable COD substrate requires energy. The phosphorus-storing microorganisms obtain this energy from the use of high-energy polyphosphate bonds (long chains of phosphates) stored in the cells.

Within the following aerobic zone, the organic polymers stored under anaerobic conditions are used by the microorganisms as a carbon source. The energy produced is stored within the cells by uptake of the wastewater phosphates and the formation of polyphosphate bonds. Thus, the phosphates from the wastewater are accumulated in the phosphorus-storing microorganism cells, and the sludge produced is removed from the treatment plant with a high phosphorus content.

The main requirements for EBPR include the presence and availability of readily biodegradable COD substrate in the anaerobic zone and an alternate exposure of the sludge biomass to anaerobic and aerobic conditions.

The presence of nitrates is known to cause important interference with the EBPR process. EBPR has been reported to be achieved with SRTs ranging from 3 to 68 days (WEF, 1998). However, operation at long SRTs may decrease the efficiency of phosphorus removal (USEPA, 1987). The EBPR process becomes stable at SRTs as low as 4 days, but at SRTs of 6 days and temperatures above 15°C, nitrification may occur within the aerobic zone (WEF, 1998) and produce nitrates. The nitrates that are recycled with the RAS in the front-end anaerobic zone favour the growth of denitrifying organisms that use the readily biodegradable COD substrate as their carbon source in place of the phosphorus-storing bacteria. In these conditions, a small portion, if any, of readily biodegradable COD substrate is available to the phosphorus-storing bacteria in the anaerobic zone. This nitrate interference reduces the EBPR process efficiency.

Control and reduction of nitrates generated from the nitrification process must be included in EBPR configurations to avoid or reduce the nitrate interference. Two major approaches have been proposed and used to address the control of nitrates. The first approach is based on the denitrification of the RAS, and the second is based on the denitrification of the nitrified mixed liquor prior to secondary clarification.

Denitrification of the RAS includes the following configurations:

- Anoxic cell located on the sludge recycle line, prior to the front-end anaerobic zone. The carbon source for the denitrification process originates from the endogenous decay. Once the RAS has been denitrified in the anoxic zone, it enters the front-end anaerobic zone for the first step of the EBPR process. A typical application of this configuration is the Johannesburg Process (Comeau, 1990) and is shown in Figure 5.21.
- Anoxic cell located between anaerobic and aerobic zones on the main liquid train. The carbon source for the denitrification process originates from the wastewater effluent/mixed liquor leaving the anaerobic zone. Once the RAS has been denitrified in the anoxic zone, the denitrified mixed liquor is recycled to the front-end anaerobic zone for the first step of the EBPR process. A typical application of this configuration is the high-rate Phoredox process (Randall *et al.*, 1992) and is shown in Figure 5.22.

Denitrification of the nitrified mixed liquor prior to secondary clarification involves the internal

**Figure 5.21 Johannesburg Process Schematic**

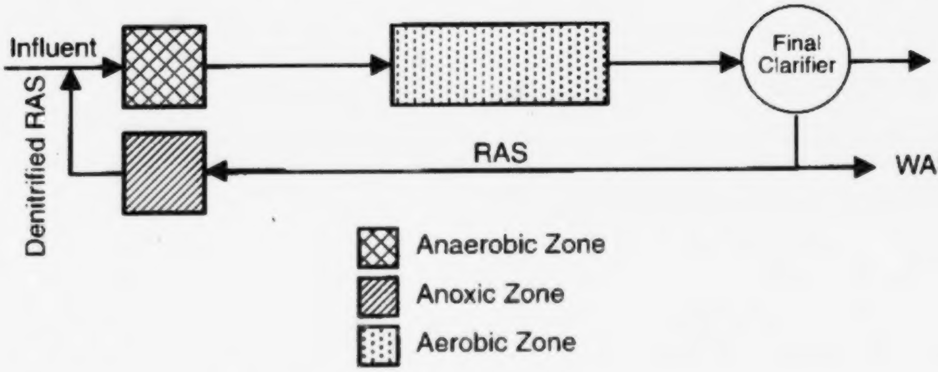
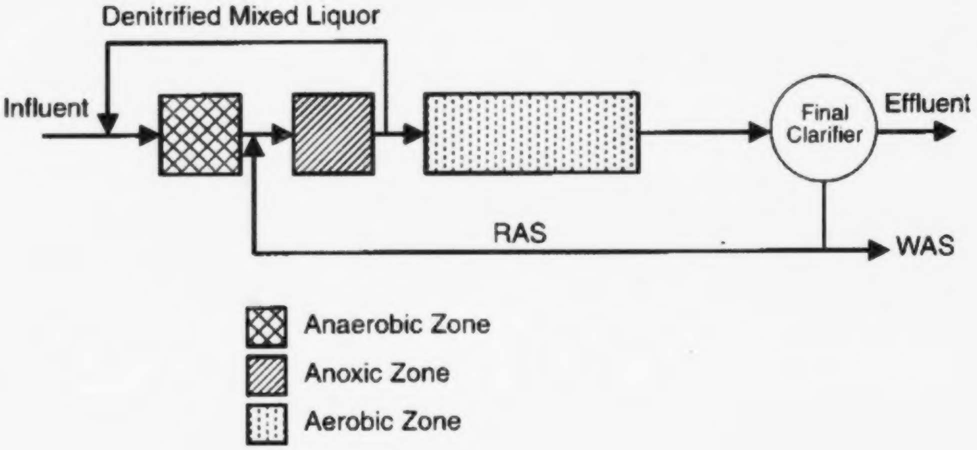

Influent
Final Clarifier
Denitrified RAS
RAS
WAS
Anaerobic Zone
Anoxic Zone
Aerobic Zone

**Figure 5.22 High-Rate Phoredox Process Schematic**


Denitrified Mixed Liquor
Influent
Final Clarifier
Effluent
RAS
WAS
Anaerobic Zone
Anoxic Zone
Aerobic Zone

recycling of the nitrified mixed liquor in an anoxic zone located upstream of the aerated zone. The carbon source for the denitrification process originates from the wastewater effluent/mixed liquor leaving either the anaerobic zone or a preceding anoxic zone. This configuration permits reduction of the nitrate content of the mixed liquor before clarification and thus reduces the nitrate content of the RAS. A typical application of this configuration is the five-stage Bardenpho process (USEPA, 1987; Comeau, 1990; WEF and ASCE, 1998).

Other process variations that combine the two configurations, RAS and nitrified mixed liquor denitrification, for the control and reduction of nitrates include the University of Cape Town and modified University of Cape Town processes, depicted in Figures 5.23 and 5.24, respectively (Comeau, 1990; WEF and ASCE, 1998).

The major advantages for the EBPR process include the following:

- elimination/reduction of chemical addition for phosphorus removal;
- reduction of aeration requirement and equipment;
- cost reduction in terms of chemical additions and aeration requirement;
- improved nitrification rates in the aerobic zone by removing part of the soluble organic substrate in the anaerobic zone;
- reduction of excess sludge generation, as a result of elimination/reduction of chemical addition;
- facilitation of sludge processing;
- excess sludge more amenable to land application; and
- achievement of total nitrogen removal with the EBPR process.

### 5.3.2.2 Chemical Phosphorus Removal

Chemical phosphorus removal may be part of the primary, secondary or tertiary process. The basic principle of chemical phosphorus removal is the transformation of soluble phosphorus to a particulate form by chemical precipitation and flocculation. The treatment is followed by sedimentation, flotation or filtration of the precipitates.

Chemical precipitation of phosphorus from wastewater requires the addition of a coagulant. Under the right conditions, the coagulant forms insoluble precipitates with a phosphorus content, which can be removed and disposed of. Coagulants typically used for phosphorus precipitation include the following:

- aluminum
    - alum ($Al_2(SO_4)_3 \cdot 14H_2O$)
    - sodium aluminate ($NaAlO_2$)
    - polyaluminum chloride

- iron
    - ferric chloride ($FeCl_3$)
    - ferrous chloride ($FeCl_2$)
    - ferric sulphate ($Fe_2(SO_4)_3$)

- calcium
    - lime ($Ca(OH)_2$)

The efficiency of chemical phosphorus removal depends on three major parameters: the coagulant dosing requirements, the effects of pH and the minimum phosphate concentration achievable. More information on chemical phosphorus removal may be found in references such as USEPA (1987), Metcalf & Eddy (1991), WEF (1998) and WEF and ASCE (1998).

Chemical phosphorus removal may have an impact on the alkalinity concentration of the wastewater. Wastewater alkalinity is the chief variable that affects phosphorus removal by lime. The dose of lime and the amount of solids produced are functions of the wastewater alkalinity. Alum and ferric chloride consume alkalinity with side reactions. In such cases, alkalinity concentration may be reduced, and less alkalinity will be available for nitrification process buffering capacity requirements or to maintain a minimal residual alkalinity. Thus, alkalinity supplementation must be provided following a severe reduction within the phosphorus chemical precipitation step.

**Figure 5.23 University of Cape Town Process Schematic**

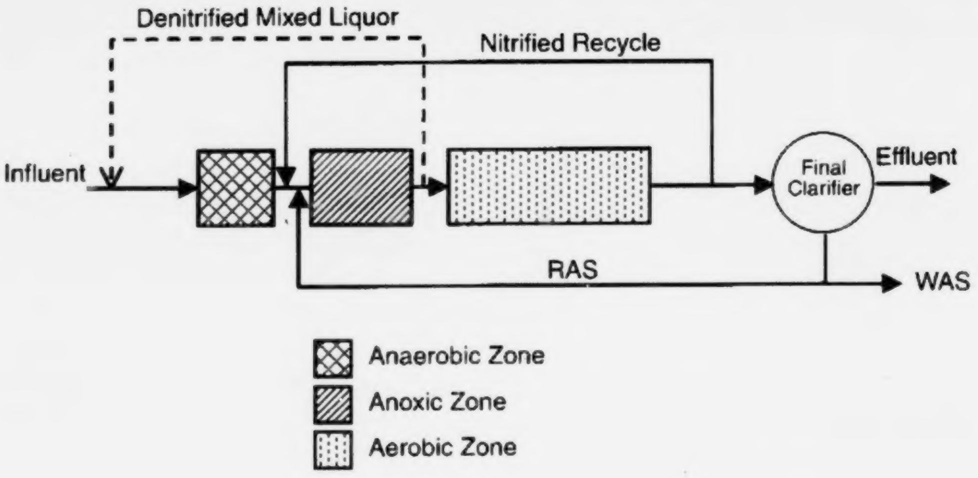
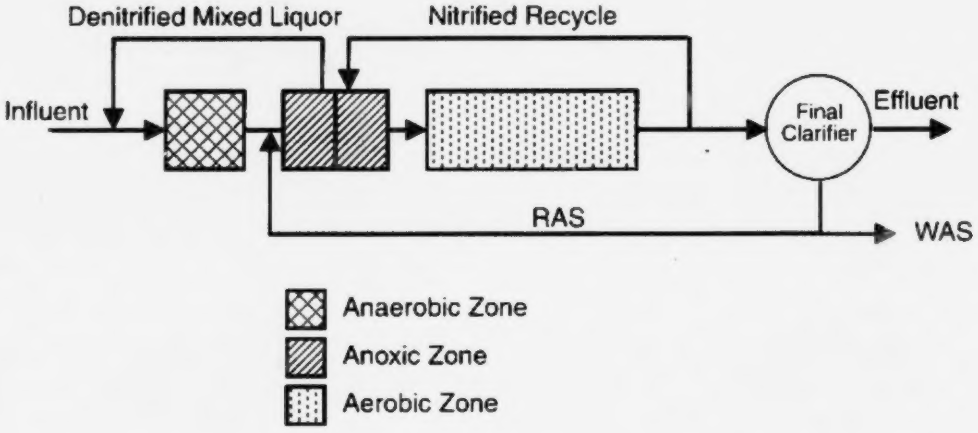

Denitrified Mixed Liquor
Nitrified Recycle
Influent
Final Clarifier
Effluent
RAS
WAS
Anaerobic Zone
Anoxic Zone
Aerobic Zone

**Figure 5.24 Modified University of Cape Town Process Schematic**


Denitrified Mixed Liquor
Nitrified Recycle
Influent
Final Clarifier
Effluent
RAS
WAS
Anaerobic Zone
Anoxic Zone
Aerobic Zone

## 5.4 Innovative Technologies for Ammonia Removal

A number of promising new processes continue to be developed for the removal of nutrients from municipal wastewaters. These include novel biological nutrient removal process configurations incorporating both suspended-growth and attached-growth biomass optimized for traditional aerobic autotrophic nitrification, pre- or postdenitrification and/or EBPR. There are experimental investigations and developments in oxidation ditch process optimization to promote heterotrophic nitrification coupled with aerobic denitrification, which may offer certain advantages. These processes are described below, as they are still in the early development stage with no or only few full-scale demonstrations.

## *5.4.1 Biological Technologies*

### 5.4.1.1 Partial Nitrification-Denitrification — The Sharon Process

Achieving partial nitrification from ammonia to nitrite followed by denitrification is interesting for its potential of reducing aeration energy for nitrification and COD addition for denitrification. Attempts were conducted with a laboratory-scale activated sludge process using anoxic and aerobic zones by controlling a number of environmental conditions (e.g., inhibition by free ammonia, nitrous acid or chlorate; nitrite reduction; low MLSS), which were successful, but only for a limited duration (i.e., never more than 125 days), due to biomass acclimation (Turk and Mavinic, 1989).

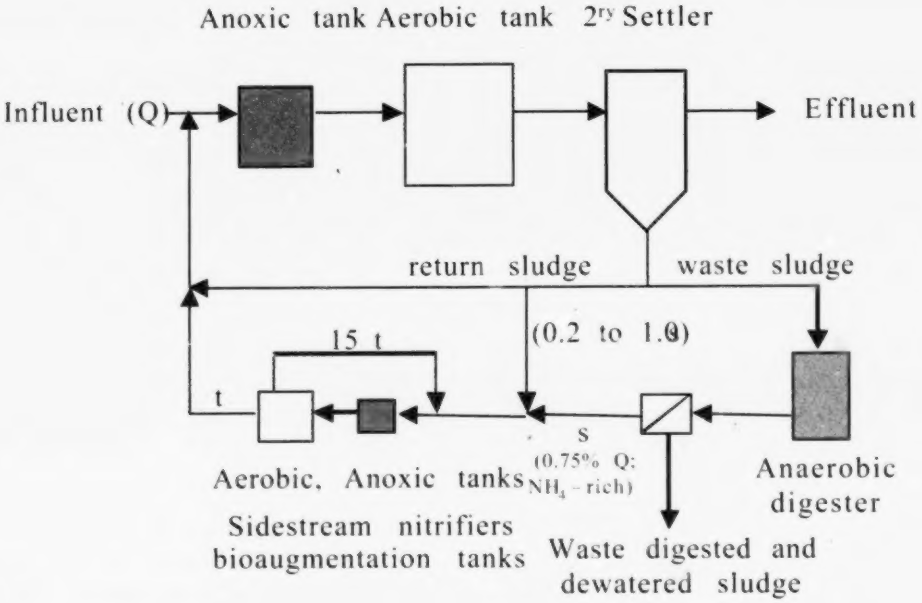
Partial nitrification-denitrification can, however, be achieved in the Sharon (Single reactor system for High activity Ammonia Removal Over Nitrite) process, which was developed to treat ammonia-rich streams, such as anaerobic digester supernatant, by operating a single well-mixed reactor at high temperature (30–40°C) without sludge retention. Cycles of 80 minutes of aeration and 40 minutes of denitrification (without aeration) are used to achieve partial nitrification to nitrite (instead of nitrate) and denitrification to nitrogen gas, thus reducing aeration energy (25%) and COD addition (e.g., methanol; 40% savings) costs. Partial nitrification is achieved by exploiting the fact that at a temperature above 13°C, the growth rate of ammonia oxidizers is greater than that of nitrite oxidizers. The total HRT, which equals the SRT, is set to 1.5 days (Hellinga *et al.*, 1998). The process has been tested successfully at full scale (Mulder *et al.*, 2001).

### 5.4.1.2 The Babe Process

The Babe process achieves nitrifier bioaugmentation in a sidestream reactor including an anoxic and an aerobic tank (Figure 5.25). An ammonia-rich stream (e.g., process water from an anaerobic sludge digester) is fed into the sidestream process, which has an HRT equal to its SRT of 1.3–2.2 days. No settling tank is provided in this sidestream process, and the effluent is returned into the mainstream process. In the study of Salem *et al.* (2002), the flow rate of the influent wastewater was 43 200 $m^3$/d ("Q"), and the flow rate of the ammonium-rich reject water from the dewatering of the waste digested sludge was 0.0075 times "Q" ("s"). For sidestream nitrifier bioaugmentation, a fraction of the return sludge (0.2–1.0 times "s") was added to this ammonium-rich stream in a series of anoxic and aerobic tanks (effluent flow rate of "t"), with denitrification being achieved in the anoxic tank to recover alkalinity.

Providing a sidestream nitrification bioaugmentation process allows the total SRT of the process to be reduced, an interesting concept for upgrading, for designing smaller bioreactors or for nitrifying under low temperature conditions. A computer simulation of a full-scale MWWTP indicated that the nitrifying biomass was approximately doubled with this process, leading to a bioreactor volume that could be as much as two times smaller due to nitrification and denitrification taking place in the sidestream tanks (Salem *et al.*, 2002).

**Figure 5.25 Babe Process for Nitrifier Bioaugmentation in a Sidestream Reactor (adapted from Salem *et al.*, 2002)**


Anoxic tank
Aerobic tank
2ry Settler
Influent (Q)
Effluent
return sludge
waste sludge
(0.2 to 1.0)
15 t
t
S
(0.75% Q: NH4 - rich)
Aerobic, Anoxic tanks
Sidestream nitrifiers bioaugmentation tanks
Anaerobic digester
Waste digested and dewatered sludge

### 5.4.1.3 Dephanox Process

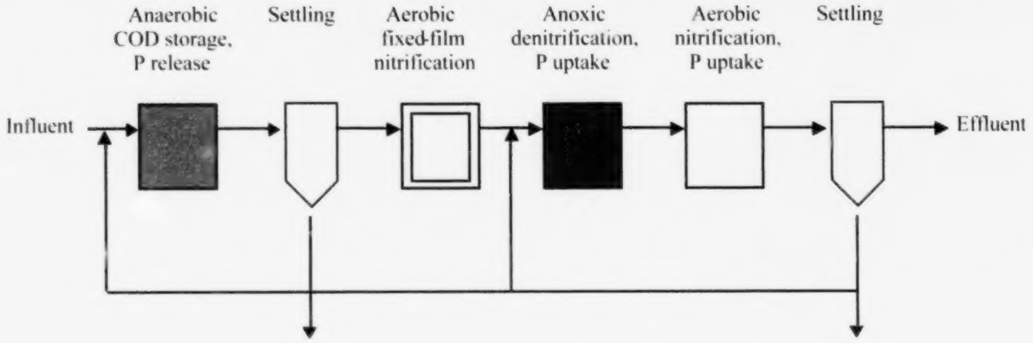
The Dephanox process exploits the use of nitrate, instead of oxygen, for phosphate uptake in an EBPR process. After the standard anaerobic zone where influent and return sludge are combined to favour organic matter adsorption and storage by the biomass, a clarifier is inserted into the process to recover the ammonia-containing supernatant to send it to a nitrifying fixed film while the sludge is sent to the downstream reactor where nitrate is used for phosphate uptake. A final aerobic zone completes the process (Figure 5.26) (Wanner *et al.*, 1992; Sorm *et al.*, 1996). This process reduces considerably both the required SRT for stable nitrification and the oxygen demand and provides improved sludge settleability (Hu *et al.*, 2000).

### 5.4.1.4 Circox Airlift Biofilm Reactor

The Circox process uses an airlift system to circulate sand on which a biofilm develops to achieve both nitrification and denitrification (Figure 5.27). The influent is fed into a central zone, where it is mixed with biofilm particles recirculated from the aerobic zone. This "mixed liquor" is brought downward and into the aerobic zone by means of an airlift riser. A second airlift system provides recirculation into the aerobic zone. Excess liquid flows into the settler zone, where the particles are separated from the effluent.

This process can withstand high COD and nitrogen loading rates, resulting in a compact process with a very small footprint (about 10% of

**Figure 5.26 Dephanox Process for Biological Nitrogen and Phosphorus Removal**


Anaerobic COD storage, P release
Settling
Aerobic fixed-film nitrification
Anoxic denitrification, P uptake
Aerobic nitrification, P uptake
Settling
Influent
Effluent

an activated sludge process), and has been applied for both municipal (Fritjers *et al.*, 1997) and industrial wastewater treatment (Mulder and Bruijn, 1993; Fritjers *et al.*, 2000), at both pilot scale (3 $m^3$) and full scale (e.g., 130 $m^3$). Removals of 2.5 kg COD/$m^3$ per day and 0.8 kg ammonium-nitrogen per aerobic cubic metre per day were achieved, representing 29% and 90% removal, respectively, for an industrial effluent (Fritjers *et al.*, 2000). A low sludge yield was observed (2–10% of the COD converted), and no backwashing was required, as an equilibrium was reached between loading rate and biomass concentration by sludge detachment. Effluent treatment by dissolved air flotation is used to capture escaping biomass. A dense (220 mL/g) and high biomass concentration (30 g biofilm VSS/L) was reported. Both nitrification and denitrification take place due to the long sludge age.

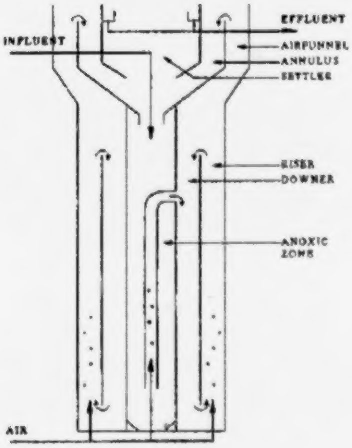
### 5.4.2 Physical/Chemical Technologies

Ammonium precipitation may take place in the form of calcium phosphates or struvite (magnesium ammonium phosphate; $MgNH_4PO_4$) pellets for nutrient recovery from municipal, industrial and agro-industry wastewaters and use as a fertilizer. High concentrations of these three chemicals are commonly found in anaerobic digester supernatants or in the anaerobic zone of an EBPR process. Fluidized-bed reactors in which aeration is used to strip carbon dioxide, thus raising the pH, have typically been used over the last 10 years for calcium phosphate or struvite formation and recovery (Battistoni *et al.*, 2000).

## 5.5 Preliminary Costing of Alternative Nitrification Processes

A summary has been prepared of estimated capital and O&M costs for alternative ammonia removal processes. These order-of-magnitude-level estimates are based on defined generic MWWTP influent conditions associated with four different wastewater treatment scenarios. The four defined scenarios for the application of new stand-alone or new add-on ammonia removal processes are outlined below.

**Figure 5.27 Schematic of the Circox Airlift Biofilm Reactor with Integrated Anoxic and Aerobic Compartments (Fritjers *et al.*, 2000)**



- *Scenario 1*: Represents the application of a new stand-alone nitrification process having an average design capacity of 25 000 m$^3$/d (5.5 MIGD) (i.e., medium-sized plant) for treatment of a medium-strength municipal wastewater. The defined process influent characteristics represented either raw wastewater or primary-treated effluent, depending on the biological treatment process assessed.
- *Scenario 2*: Represents the application of a new add-on tertiary nitrification process having an average design capacity of 25 000 m$^3$/d (5.5 MIGD) (i.e., medium-sized plant) for the posttreatment of lagoon process effluent.
- *Scenario 3*: Represents the application of a new stand-alone nitrification process having an average design capacity of 1000 m$^3$/d (0.22 MIGD) (i.e., small-sized plant) for treatment of a medium-strength municipal wastewater. This corresponds to the installation of a new nitrifying extended aeration activated sludge plant ahead of an existing lagoon-based MWWTP to ensure year-round ammonia removal.
- *Scenario 4*: Represents the application of a new add-on tertiary nitrification process having an average design capacity of 1000 m$^3$/d (0.22 MIGD) (i.e., small-sized plant) for the posttreatment of lagoon process effluent.

The defined generic influent conditions that were used as a basis for the cost estimates for the four described scenarios are outlined in Table 5.2. The selected influent characteristics are representative of a medium-strength wastewater according to Metcalf & Eddy (1991), which is considered to be conservatively high but representative, as discussed in Chapter 3. It was assumed that the wastewater contains an adequate level of alkalinity to sustain nitrification.

## Table 5.2 Generic MWWTP Influent Characteristics for Sizing and Costing of New Nitrification Process Installations

| **Process Influent Characteristics** | **Scenario** | | | | |
|---|---|---|---|---|---|
| | **1a** | **1b**[a] | **2** | **3** | **4** |
| Average flow rate ($m^3/d$) | 25 000 | 25 000 | 25 000 | 1000 | 1000 |
| Hydraulic peaking factor | 2.0 | 2.0 | 2.0 | 2.0 | 2.0 |
| $cBOD_5$ (mg/L) | 220 | 150 | 50 | 220 | 50 |
| TSS (mg/L) | 220 | 100 | 50 | 220 | 50 |
| TKN (mg/L) | 40 | 35 | – | 40 | – |
| $TKN_{hydrolyzable}$ (mg/L) | 35 | 30 | – | 35 | – |
| Total ammonia-nitrogen (mg/L) | 25 | 25 | 20 | 25 | 20 |
| Total phosphorus (mg/L) | 8 | 7 | 6 | 8 | 6 |
| pH | 7.0 | 7.0 | 7.0 | 7.0 | 7.0 |
| Temperature (°C) | 10 | 10 | 5 | 10 | 5 |

[a] Primary treated effluent for Scenario 1b was assumed to include recycle streams from solids handling.

The conservative basis of design included a wintertime temperature of 10°C for raw, preliminary or primary effluent and of 5°C for lagoon process effluent serving as influent to the nitrification process, depending on the particular scenario (Table 5.2).

Only biological nitrification processes were considered in the cost assessment, as this is considered the most feasible and economically viable option for ammonia removal from municipal wastewaters. The basis for the sizing and costing of the nitrification processes was a treated effluent total ammonia-nitrogen concentration of <5 mg/L, based on a monthly average. This discharge concentration is considered to be reasonably attainable with the nitrification processes studied.

Depending on the source of the cost information, the reported capital cost estimate may or may not represent the total construction cost (i.e., whether or not equipment installation and civil works are included). None of the capital cost estimates include other indirect project-related costs associated with engineering, administration, legal and financial services or any contingency factors. The latter indirect costs could be expected to add another 20–40%, as a percentage of the total construction cost (WEF, 1998).

All cost estimates are reported in Canadian dollars and are actualized to current values (i.e., based on the year 2000 average Engineering News-Record construction cost index of 6220). These estimates have an order-of-magnitude-level expected degree of accuracy (i.e., +50% to −30%) (WEF, 1998).

A more detailed economic assessment of different process upgrade alternatives would require a life cycle cost analysis incorporating total project cost, financing and anticipated O&M expenditures. This would be prepared by a project design consultant on a case-specific basis to provide a more complete indication of the relative economics of the alternative process options over their design life.

It is emphasized that obtaining reliable cost estimates is a complex task, with numerous factors having an influence. Many of these factors are not directly related to plant capacity, type of unit processes or desired effluent quality and are site specific and often uncontrollable and variable. A summary of various factors and design cost decisions is described in WEF (1998).

### 5.5.1 Cost Information Sources

Contacts were made with a number of major technology vendors to request process size and cost information for the defined generic applications, particularly for those processes that are considered to be non-traditional or proprietary (e.g., attached-growth systems, MBR, MBBR) or for unique activated sludge process designs, for which little reported information is available. The reporting of vendor quotations is not an endorsement of the respective processes or suppliers.

Unless otherwise indicated, the cost estimates that were obtained from vendors, based on the defined generic conditions, included only the nitrifying secondary treatment process. They did not include upstream process stages (e.g., influent pumping, preliminary and/or primary treatment or lagoon pretreatment), downstream filtration or disinfection stages, or sludge processing stages. These construction cost estimates did not include equipment installation or civil works, unless specified otherwise. O&M costs were provided by only some of the contacted suppliers.

Published cost-capacity curves and equations (USEPA, 1992; Hydromantis, Inc. and SBR Technologies Inc., 1998; WEF, 1998) were used for generating estimated costs for the more traditional treatment processes, such as activated sludge plants, biological nutrient removal systems and SBRs. The average design flow rate capacity was used as the basis for costing. In these cases, the reported total construction costs did include civil works. Furthermore, the capital and O&M costs were usually based on all of the major stages of the liquid train (e.g., wastewater pumping, preliminary and/or primary treatment, in addition to the nitrifying biological treatment process). Cost estimates were based on the average influent flow rate only.

Use was also made of the commercially available CapdetWorks™ software by Hydromantis, Inc. for preliminary treatment system sizing and costing. This PC software program enabled total construction cost and O&M costs to be estimated for the individual process stages that make up traditional treatment system trains, including pumping stations, upstream treatment stages and various nitrifying processes, such as activated sludge plants, trickling filters and RBCs.

CapdetWorks™ uses design algorithms, models, unit costing methods and an up-to-date customizable equipment and materials database to generate planning-level estimates of life cycle cost, project cost, total construction cost (i.e., including all civil works, site preparation, site electrical, yard piping, instrumentation and control, etc.) and total O&M costs (i.e., including operations, maintenance, materials, chemicals and energy at $0.10/kWh). The program used the defined design flow rate and wastewater characteristics (Table 5.2) as input parameters. Costs associated with administration and laboratory buildings and labour were not included in the reported estimates.

***Owing to the many factors that can have an impact on system designs and costs on a case-by-case basis and the different sources of information and bases that were used for the generation of the rough cost estimations reported in this section, the reader is cautioned not to draw general conclusions regarding the relative cost-effectiveness of one type of process versus another. The design requirements and costs of biological treatment processes are site specific.***

### 5.5.2 Costs for New Nitrification Process Installations

The stand-alone nitrification processes for which cost information was obtained or determined for Scenario 1 include:

- activated sludge (estimated by CapdetWorks™)
  - single-stage nitrification
  - two-stage (high-rate activated sludge plus nitrifying activated sludge processes);
- extended aeration activated sludge (estimated by CapdetWorks™);
- biological nutrient removal plant (calculated by cost-capacity equations)
  - new plant and existing plant upgrade alternatives;
- oxidation ditch (provided by vendor);
- SBR (calculated);
- MBR (vendor)
  - single-sludge nitrification plus denitrification design;
- aerated submerged fixed-bed (BAF) (vendor)
  - two-stage design;
- trickling filter (vendor and CapdetWorks™)
  - two stages, with an intermediate clarifier;
- RBC (vendor and CapdetWorks™)
  - single-stage and two-stage, with an intermediate clarifier;
- SBC (vendor)
  - single-stage without an intermediate clarifier;
- MBBR (vendor)
  - two stages without an intermediate clarifier; and
- fluidized-bed reactor (vendor)
  - tertiary nitrification.

The add-on tertiary nitrification processes for which information was obtained or predicted for Scenario 2 include:

- aerated submerged fixed-bed (BAF) (vendor);
- trickling filter (vendor);
- RBC (vendor); and
- SBC (vendor).

The stand-alone nitrification process for which cost information was determined for Scenario 3 includes:

- new extended aeration activated sludge plant (CapdetWorks™); and
- lagoon retrofit to include an extended aeration activated sludge plant (cost curves for Sutton

Process in OMOE and Environment Canada, 1993).

The add-on tertiary nitrification processes for which information was obtained under Scenario 4 include:

- trickling filter (CapdetWorks™);
- RBC (CapdetWorks™);
- intermittent sand filter (calculated and estimated from cost equations and cost curves, respectively);
- recirculating sand filter (vendor); and
- constructed wetlands (not based specifically on Scenario 4 conditions; see Section 5.5.2.1).

It is noted that in addition to achieving virtually complete nitrification, the oxidation ditch, SBR and MBR process designs can accomplish nitrate removal by denitrification owing to the presence of anoxic zones. The MBR process offers the additional benefit of producing a tertiary-treated effluent quality.

Summaries of the capital and O&M cost information obtained from vendors of the alternative nitrification processes are tabulated in Table 5.3, under the applicable scenarios. Specific remarks, including assumptions, design and cost bases, and process characteristics, are included in the last column entitled "Remarks." Cost information that was determined by published cost-capacity curves/equations or by CapdetWorks™ has been summarized in Tables 5.4 and 5.5, respectively.

CapdetWorks™ estimates of total construction cost and total annual O&M cost were used to prepare cost-capacity curves based on design average wastewater flow rates of 1 MLD, 25 MLD and 100 MLD (i.e., representing small-, medium- and large-sized plants, respectively) for the entire liquid train of a nitrifying MWWTP. Figures 5.28 and 5.29 show the total construction cost curve and the total annual O&M cost curve, respectively. The generic liquid train includes an influent pump station, preliminary and primary treatment, single-stage nitrifying activated sludge secondary treatment, interstage pumping, tertiary filtration and ultraviolet disinfection. The medium-strength wastewater characteristics defined under Scenario 1a in Table 5.2 were used as a basis.

#### 5.5.2.1 Costs of Constructed Wetlands for Lagoon Effluent Posttreatment

Another option for consideration in the posttreatment of lagoon process effluents is constructed wetlands technology. The application of this technology is highly site specific and requires engineering design by specialized consultants. Reported cost information was cited, as described below.

The major items in the capital costs for both types of constructed wetlands, FWS and VSB, should include land costs, site investigation, clearing and grubbing, excavation and earthwork, liner, media, aquatic plants, vegetation establishment, inlet structures, outlet structures, fencing, miscellaneous piping and pumps, engineering services, legal and contingencies (USEPA, 2000). The majority of the costs identified are related to the surface area design of the constructed wetland. The major capital cost differences between FWS wetlands and VSBs are for the media and the aquatic plants. Site characteristics and climate conditions significantly affect the constructed wetlands system capital costs. Topography of the area and soil conditions

**Figure 5.28** Total Construction Cost Curve for the Liquid Train of a Nitrifying Activated Sludge Process-Based MWWTP Based on CapdetWorks™ Estimates

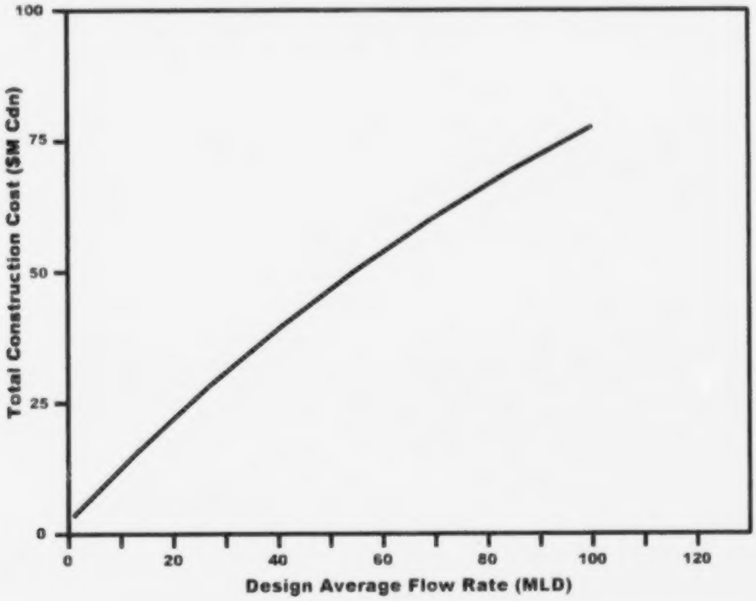
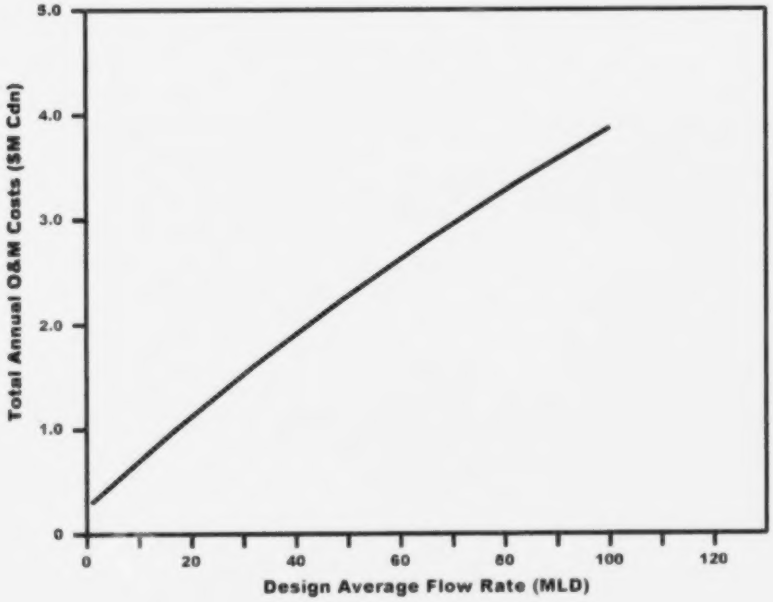


**Figure 5.29** Total Annual O&M Cost Curve for the Liquid Train of a Nitrifying Activated Sludge Process-Based MWWTP Based on CapdetWorks™ Estimates



impact the capital costs according to the amount of excavation and earthwork required. Existing lagoon retrofit in wetland systems would keep the construction work to a minimum (USEPA, 2000). Local conditions, such as availability of appropriate granular media, may influence the cost of the wetlands. A range of capital costs is proposed in this section, but should be used only as an order-of-magnitude estimation, considering that constructed wetland costs are site specific.

Operation and maintenance of constructed wetlands require limited intervention from the operator and can be compared with the operation level of facultative lagoons (USEPA, 2000). The principal items requiring the intervention of the operator include water level adjustments and flow control, maintenance of flow inlet and outlet structures, aquatic vegetation management, odour control, berm and dyke maintenance, and monitoring program management.

A range of capital and O&M costs are provided for two different flow rates in Table 5.6 (NovaTec Consultants Inc., 1996). Cost data are adjusted to current values in Canadian dollars using Engineering News-Record's construction cost index. The costs were generated for treatment of secondary lagoon effluent, including ammonia reduction. The capital costs are exclusive of land purchases, additional conveyance of effluent, contingencies and legal services (NovaTec Consultants Inc., 1996). The authors indicated that these cost data should be used cautiously and only for comparison purposes rather than for estimating actual construction costs.

The constructed wetlands can be designed with different models and calculations to achieve different water quality treatment levels. In this sense, treatment areas of wetland design at a given flow rate may vary, and capital and O&M costs will vary accordingly. Considering that constructed wetlands are site specific, it is advisable to perform construction and O&M cost estimation for each wetland system upgrade application.

### 5.5.3 Costs for Retrofit Process Alternatives for Total Nitrogen Removal

The retrofitting of existing activated sludge plants with innovative hybrid technologies for enhanced treatment and nutrient removal represents a cost-effective upgrade option. A recent report (WERF, 2000) provided a summary of such an upgrade based on a generic 38 MLD activated sludge plant, retrofitted with different attached-growth support media or immersed membrane separation systems for enhanced nitrification. Various IFAS-based hybrid process configurations, an MBBR attached-growth process and an MBR suspended-growth process configuration were comparatively assessed. The basis for the design was to achieve complete nitrification and total nitrogen concentrations of <8 mg/L, without expansion of the existing aeration tank volume of the generic treatment facility.

Preliminary capital cost estimations were reported for five specific retrofit cases that were investigated (WERF, 2000):

- IFAS process configurations involving the addition of rope media (Ringlace), sponge media (Linpor from Lotepro) or free-moving plastic media (Kaldnes);

## Table 5.3 Cost of Ammonia Removal Technologies (Vendor Information)

| | Capital and O&M Costs | | | | |
|---|---|---|---|---|---|
| **Process** | **Scenario 1a** | **Scenario 1b** | **Scenario 2** | **Scenario 3** | **Scenario 4** |
| *Biological Suspended Growth* | | | | | |
| Activated sludge<br>(i.e., single-stage extended aeration process)<br>BIOLAC<br>[Parkson Corp.] | Capital cost:<br>9,240,000 $[2]<br>O&M cost:<br>not available | | | | |
| Membrane bioreactor (MBR)<br>(i.e., immersed hollow fibre membrane based; extended aeration with anoxic zone)<br>[Zenon Environmental] | | Capital cost:<br>15,000,000 $<br>O&M cost:<br>650,000 $/year | | | |
| Oxidation ditch<br>(extended aeration activated sludge; triple ditch design; SBR mode)<br>[USFilter Kruger Products] | Capital cost:<br>1,330,000 $[2]<br>Concrete work:<br>1,600,000 $[2]<br>O&M cost:<br>470,000 $/year | | | | |
| Trickling filter (TF)<br>[Mass Transfer] | Capital cost:<br>5,360,000 $[1]<br>O&M cost:<br>75,000 $/year | Capital cost:<br>3,540,000 $[1]<br>O&M cost:<br>72,500 $/year | Capital cost:<br>4,300,000 $[1]<br>O&M cost:<br>75,000 $/year | | |
| *Biological Suspended Growth* | | | | | |
| Rotating biological contactor (RBC)<br>[USFilter (Envirex) / John Meunier] | Capital cost:<br>7,120,000 $[2]<br>O&M cost:<br>311,000 $/year | Capital cost:<br>6,900,000 $[2]<br>O&M cost:<br>259,000 $/year | Capital cost:<br>4,180,000 $[2]<br>O&M cost:<br>156,000 $/year | | |
| Submerged biological contactor (SBC)<br>[USFilter (Envirex) / John Meunier] | Capital cost:<br>8,930,000 $[2]<br>O&M cost:<br>338,000 $/year | Capital cost:<br>7,440,000 $[2]<br>O&M cost:<br>282,000 $/year | Capital cost:<br>4,460,000 $[2]<br>O&M cost:<br>171,000 $/year | | |

| Remarks |
|---|
| *Scenario 1a*: Two parallel trains, each including a Biolac aeration basin (55-day SRT) with 4.6-m SW depth and 15-ML volume.<br>Each basin comes with an integral InterClar secondary clarifier with weir, rake and 825 SCFM airlift sludge return pump.<br>Capital cost includes 4735 SCFM BioFlex aeration system with BioFuser devices and control valves, air supply piping, three 150 HP blowers, control panel with motor starters, switches and timers. All equipment installed and civil works plus inspection, start-up supervision and training included.<br>nfluent pumping and pretreatment, DO measurement/monitoring are not included. |
| *Scenario 1b*: Four parallel aeration tanks (>15-day SRT) with 4.5-m SWD. Each aeration tank contains a 25% v/v upfront anoxic zone. Each aeration tank contains 30 membrane cassettes with 446 $m^2$ of surface area per cassette (module configuration: ZW500-C) for a total surface area of 53 500 $m^2$. Hollow fibre membranes are ultrafilters with nominal pore size of 0.035 µm. Each cassette is aerated on a 10-second on/off interval at an airflow rate of 200 SCFM per cassette.<br>Capital costs include membranes, stainless steel membrane support frames, permeate headers, permeate pumps, recirculation pumps, fine-bubble diffusers, sludge wasting pumps, blowers, VFDs and motor starters (but excluding MCC), PLC, instrumentation, controls, on-site installation, commissioning supervision and operator training. Influent pumping station, preliminary treatment, ncluding fine screening, primary treatment and sludge treatment are not included. Civil works, freight, contractor fees and taxes are not included.<br>O&M costs are based on membrane replacement, chemicals and electrical consumption. Manpower is not included. |
| *Scenario 1a*: One train with three interconnected oxidation ditches. No secondary clarification stage required. Each oxidation ditch is 65 m in length, 22 m in width, 11-m channel width, 4-m side water depth. Three influent weirs and 10 effluent weirs are installed per train. Four brush aerators (rotors) are installed per ditch.<br>Capital costs include brush aerators (rotors), influent and effluent weirs, guiding masts for future installation of submersible mixers, nstrumentation and control system with PLC panel and field instrumentation (DO probes and ultrasonic sensors), field services ncluding installation inspection, initial start-up operation assistance and operation training.<br>nfluent pumping station, preliminary and primary treatment, and sludge treatment are not included. All civil works including water and wastewater piping, interconnecting electrical wiring and conduit, installation and supervision, motor control equipment, field erminations, power distribution equipment, motor starters, contactors, reversing contactors, local control panels and junction boxes are not included.<br>O&M costs are based on brush aerators (rotors) electrical consumption. Manpower is not included. |
| *Scenario 1a*: Two parallel trains with two TFs per train. First-stage TFs are 681 $m^2$ each (carbonaceous removal), 6.1-m media depth, no recirculation required. Second-stage TFs are 461 $m^2$ each (nitrification), 6.1-m media depth, no effluent recirculation required. nterstage clarifiers are required between the first and second stages.<br>*Scenario 1b*: Two parallel trains with two TFs per train. First-stage TFs are 328 $m^2$ (carbonaceous removal), 6.1-m media depth, no recirculation required. Second-stage TFs are 393 $m^2$ each (nitrification), 6.1-m media depth, no effluent recirculation required. nterstage clarifiers are required between the first and second stages.<br>*Scenario 2*: Two parallel trains with one TF per train. TFs are 768 $m^2$ each (carbonaceous removal and tertiary nitrification), 6.1-m media depth, effluent recirculation is required with a ratio of 1:1 ($Q_{rec.}/Q_{infl.}$).<br>*For all scenarios*:<br>Capital costs are for stand-alone TFs including civil structures, speed-controlled rotary distributors, fans, media material, installation, electricity and civil work. Influent pumping station, preliminary and primary treatment, interstage and final clarifiers, lift stations, treated water recirculation pumps, optional sand filtration stage for polishing and sludge treatment are not included. The capital costs for the stand-alone TFs are based on a factor of 2.5 × the media cost, as per the manufacturer indication. The factor will vary according to he ground condition, tank shell material, etc.<br>O&M costs are based on energy consumption from the influent pumping station to the TFs, TF units influent rotary distributors, fan operation and lift station operation between the first- and second-stage TFs (Scenarios 1a and 1b) or one treated water ecirculating pump (Scenario 2). Manpower is not included. |
| *Scenario 1a*: 257 523 $m^2$ for carbonaceous removal and 754 610 $m^2$ for nitrification, 72 RBC shafts total. Single-stage design with no interstage clarifier.<br>*Scenario 1b*: 146 320 $m^2$ for carbonaceous removal and 644 640 $m^2$ for nitrification, 60 RBC shafts total. Single-stage design with no interstage clarifier.<br>Scenario 2: 73 168 $m^2$ for carbonaceous removal and 424 875 $m^2$ for nitrification (tertiary nitrification), 36 RBC shafts total. Single-stage design with no interstage clarifier.<br>*For all scenarios*:<br>All RBC equipment designs include low-, medium- and high-density media and do not include *supplemental* aeration.<br>Capital costs include complete RBCs with media supports, shaft drives and FRP covers. All civil and electrical work and installation are not included. Influent pumping station, preliminary and primary treatment, final clarifiers and sludge treatment are not included. No dedicated influent pumping station would likely be required for scenario 2.<br>O&M costs are based on electricity consumption by RBC drives and manpower. |
| *Scenario 1a*: 257 523 $m^2$ for carbonaceous removal and 754 610 $m^2$ for nitrification, 36 SBC shafts total. Single-stage design with no nterstage clarifier.<br>*Scenario 1b*: 146 320 $m^2$ for carbonaceous removal and 644 640 $m^2$ for nitrification, 30 SBC shafts total. Single-stage design with no nterstage clarifier.<br>*Scenario 2*: 73 168 $m^2$ for carbonaceous removal and 424 875 $m^2$ for nitrification (tertiary nitrification), 18 SBC shafts total. Single-stage design with no interstage clarifier. |

## Table 5.3 Cost of Ammonia Removal Technologies (Vendor Information) *Contd.*

| | | | Capital and O&M Costs | | |
|---|---|---|---|---|---|
| **Process** | **Scenario 1a** | **Scenario 1b** | **Scenario 2** | **Scenario 3** | **Scenario 4** |
| Aerated submerged fixed bed (BAF)<br>Biofor<br>[Degrémont] | | Capital cost: 2,500,000 $<br>O&M cost: not available | | | |
| Aerated submerged fixed bed (BAF)<br>Biostyr<br>[OTV / John Meunier] | | Capital cost: 1,960,000 $ [3]<br>O&M cost: equivalent to conventional activated sludge with nitrification | Capital cost: 982,000 $ [3]<br>O&M cost: equivalent to conventional activated sludge with nitrification | | |
| Aerobic submerged mobile bed (fluidized-bed reactor)<br>Biolift<br>[OTV / John Meunier] | | Capital cost: 6,400,000 $ [3]<br>O&M cost: 410,000 $/year | | | |
| Recirculating sand filter (RSF)<br>[Orenco] | | | | | Capital cost: 2,940,000 $ [2]<br>O&M cost: 59,000 $/year |
| Aerobic submerged mobile bed, moving-bed biofilm reactor (MBBR)<br>[Kaldnes] | | Capital cost: 3,180,000 $ [2]<br>O&M cost: 366,000 $/year | | | |

Abbreviations used: FRP = fiberglass reinforced polymer; HP = horsepower; MCC = motor central centre; PLC = programmable logi
SCFM = standard cubic feet perminate; SWD = side water depth; VFD = variable-frequency drive.

[1] Conversion factor: 1 pound sterling = 2.23 Cdn. dollars.
[2] Conversion factor: 1 U.S. dollar = 1.55 Cdn. dollars.
[3] Conversion factor: 1 French franc = 0.215 Cdn. dollar.

| Remarks |
|---|
| *For all scenarios*:<br>All SBC equipment designs include high-density media, process and drive air headers.<br>Capital costs include complete SBCs with media support and FRP covers. All civil and electrical work and installation are not included. Blowers are not included. Influent pumping station, preliminary and primary treatment, final clarifiers and sludge treatment are not included. No dedicated influent pumping station would likely be required for scenario 2.<br>O&M costs are based on electricity consumption by SBC air requirements and manpower. |
| *Scenario 1b*: Four parallel trains with two Biofor units per train. First stage includes four Biofor C cells for carbonaceous removal of 40 $m^2$ each with 2.5-m depth of biolite N media of 2.7-mm effective size. Following second stage includes four Biofor N cells for nitrification with the same characteristics.<br>Capital cost includes the fine screening, the eight Biofor cells internal component equipment (including the nozzle floor, the air headers and the biolite), blowers and pumps for the air scouring and backwash requirements, and the control system. Process blowers are not included. All civil and electrical work, installation and interconnecting piping are not included. Influent pumping station, primary treatment and sludge treatment are not included. |
| *Scenario 1b*: Three parallel trains with two Biostyr units per train. First stage includes three Biostyr cells for carbonaceous removal of 84 $m^2$ each with 3.5-m depth of biostyrene medium. Following second stage includes three Biostyr cells for nitrification with the same characteristics.<br>*Scenario 2*: Four Biostyr cells in parallel for tertiary nitrification of 63 $m^2$ each with 3.0-m depth of biostyrene medium.<br>*For all scenarios*:<br>Capital cost includes the Biostyr cells internal component equipment (including the nozzle ceiling and the biostyrene media), control valves, process/backwash blowers, and the control system. All civil and electrical work, installation and interconnecting piping are not included. Influent pumping station, preliminary and primary treatment (including screening), and sludge treatment are not included. |
| *Scenario 1b*: Biolift design characteristics are restricted to an existing conventional activated sludge upgrade for tertiary nitrification add-on system.<br>Capital cost is for a complete Biolift reactor including civil structures, lift station, airlift for fluidization and required air blower, media, process air blowers, air headers, degas system, mixed liquor recirculation line for nitrate reduction and pumps, installation, electricity and civil work. Capital cost is based on manufacturer's ratio of 77$/inhabitant equivalent.<br>O&M costs based on electricity consumption from fluidization and process air blowers and pumping requirements. Electricity consumption is based on manufacturer's ratio of 0.45 KWh/$m^3$ of treated water. Manpower is not included. |
| *Scenario 4*: Five parallel RSF cells of 200 $m^3$/d each, 0.60-m granular media filter depth, 1.5- to 2.5-mm granular media size, 0.40 $m^3/m^2$ per day hydraulic loading rate, 5:1 recirculating ratio.<br>Capital cost is an estimated installed cost including controls, pumps, media, engineering, permits, labour, etc. Manufacturer indicates that the two major factors affecting the costs are local media and labour. The design and capital cost are based on a stand-alone RSF process application downstream of an existing continuous-discharge lagoon process.<br>O&M cost includes electricity requirements. |
| *Scenario 1b*: Four parallel process trains with two MBBR reactors in series (two stages) per train. First-stage and second-stage reactors are 765 $m^3$ each with 6.4-m side water depth. Each reactor has a 40% volume fill of K1 medium. No interstage clarifiers are required. All biomass and solids generated in the aerobic reactors will flow from the last MBBR reactor to the secondary clarifier for settling out and disposal. No RAS from the secondary clarification to the MBBR reactors is required.<br>Capital cost includes 2420 $m^3$ of Kaldness K1 medium, stainless steel coarse-bubble aeration system in separate grid sections for a total of 8000 SCFM requirement, 32 stainless steel sieve assemblies for retaining the media in each of the MBBR reactors. All civil and electrical work and installation are not included. Influent pumping station, preliminary and primary treatment (including screening) clarifiers, blowers, tankage and sludge treatment are not included.<br>O&M costs are based only on electricity consumption by blowers to provide process air requirements. Manpower is not included. |

ontroller; $Q_{infl.}$ = influent flow; $Q_{rec.}$ = recirculated flow;

## Table 5.4 Cost of Ammonia Removal Technologies (Based on Cost Equations/Curves)

| | Capital and O&M Costs | | | | |
|---|---|---|---|---|---|
| **Process** | **Scenario 1a** | **Scenario 1b** | **Scenario 2** | **Scenario 3** | **Scenario 4** |
| New biological nutrient removal (BNR) plant<br>[WEF, 1998] | Capital cost:<br>36,200,000 $[1]<br>O&M cost:<br>4,000,000 $/year | | | | |
| Retrofitted BNR (activated sludge plant upgrade, without expansion)<br>[WEF, 1998] | Capital cost:<br>17,900,000 $[1]<br>O&M cost:<br>not available | | | | |
| Sequencing batch reactor (SBR)<br>[Hydromantis, Inc., and SBR Technologies Inc., 1998] | Capital cost:<br>17,400,000 $[1]<br>O&M cost:<br>not available | | | | |
| Activated sludge (i.e., extended aeration plant; lagoon retrofit)<br>[OMOE and Environment Canada, 1993] | | | | Capital cost:<br>1,850,000 $<br>O&M cost:<br>115,000 $/year | |
| Intermittent sand filter (ISF)<br>[OMOE and Environment Canada, 1993] | | | | | Capital cost:<br>725,000 $<br>O&M cost:<br>60,000 $/year |
| Intermittent sand filter (ISF)<br>[USEPA, 1992] | | | | | Capital cost:<br>530,000 $[1] |

[1] Conversion factor: 1 dollar U.S. = 1.55 Cdn. dollars.

| Remarks |
|---|
| *Scenario 1a*: Capital and O&M costs calculated from a reported cost-capacity equation. The capital cost value represented total construction cost, including all civil works but without indirect costs. The reported information was based on a survey of numerous new full-scale BNR installations in the early 1990s. The reported costs included all associated liquid train components. They did not include the solids train processes.<br>No influent characterization data were provided as a basis. |
| *Scenario 1a*: Capital cost calculated from a reported cost-capacity equation. This value represented total construction cost, including all civil works, but without indirect costs. The reported information was based on a survey of numerous full-scale BNR upgrades in the early 1990s. Costs included all associated liquid train components. They did not include the solids train processes.<br>No influent characterization data were provided as a basis. |
| *Scenario 1a*: Capital cost determined from a reported cost-capacity curve. This value represented total construction cost, including all civil works, but without indirect costs. The reported information was based on a survey of numerous new SBR installations that were reportedly capable of achieving effluent total nitrogen levels of <5 mg/L. Costs included all associated liquid train components, which did not include primary clarifiers (i.e., unnecessary). No solids train processes were included. |
| *Scenario 3*: Capital and O&M costs obtained from reported cost-capacity curves. Capital cost represents total construction cost, including civil works, but excluding indirect costs. Includes preliminary treatment and the nitrification process, but no influent pumping. The use of concrete tanks for the aeration cells was assumed. The downstream polishing lagoon of the Sutton Process was not included in costing.<br>Virtually complete nitrification was assumed as a basis for the sizing and costing of the extended aeration process. |
| *Scenario 4*: Capital and O&M costs obtained from reported cost-capacity curves. This is based on a generic stand-alone ISF process application downstream of an existing seasonal lagoon process with 180 days of winter storage capacity. The corresponding ISF process filtration area would be 1800 $m^2$.<br>No ISF influent characterization data or temperature bases were provided.<br>A monthly average treated effluent total ammonia-nitrogen limit of 4 mg/L was used as a basis. |
| Scenario 4: Capital cost calculated from a reported cost-capacity equation. The cost equation was reported to be based on a very limited database but was considered to be conservative. It is presumed that the cost is for a stand-alone ISF process (i.e., not including upstream components).<br>No influent characterization data or temperature bases were provided. |

**Table 5.5 Cost of Ammonia Removal Technologies (Estimated by CapdetWorks™)**[1]

| | Capital and O&M Costs | | | | |
|---|---|---|---|---|---|
| **Process** | **Scenario 1a** | **Scenario 1b** | **Scenario 2** | **Scenario 3** | **Scenario 4** |
| *Suspended Growth* | | | | | |
| Activated sludge<br>(i.e., single-stage nitrification) | | Capital cost:<br>15,500,000 $<br>O&M cost:<br>545,000 $/year | | | |
| Activated sludge<br>(i.e., two-stage nitrification including high-rate activated sludge plus nitrifying activated sludge) | | Capital cost:<br>15,200,000 $<br>O&M cost:<br>670,000 $/year | | | |
| Activated sludge<br>(i.e., extended aeration plant) | Capital cost:<br>20,300,000 $<br>O&M cost:<br>935,000 $/year | | | | |
| Activated sludge<br>(i.e., extended aeration plant) | | | | Capital cost:<br>2,160,000 $<br>O&M cost:<br>165,000 $/year | |
| *Attached Growth* | | | | | |
| Trickling filter (TF)<br>(two-stage nitrification including two TFs for Scenario 1b)<br>(single-stage add-on nitrifying TF for Scenario 4) | | Capital cost:<br>22,200,000 $<br>O&M cost:<br>575,000 $/year | | | Capital cost:<br>1,680,000 $<br>O&M cost:<br>97,000 $/year |
| Rotating biological contactor (RBC)<br>(two-stage nitrification including two RBCs for Scenario 1b)<br>(single-stage add-on nitrifying RBC for Scenario 4) | | Capital cost:<br>19,800,000 $<br>O&M cost:<br>525,000 $/year | | | Capital cost:<br>1,650,000 $<br>O&M cost:<br>75,000 $/year |

---

[1] Conversion factor: 1 U.S. dollar = 1.55 Cdn. dollars.

| Remarks |
|---|
| *Scenario 1b*: Total construction cost, including civil works, site preparation, piping, electrical, instrumentation and control. Includes only the nitrification process stage. O&M costs do not include administration and laboratory labour.<br>Influent pump station, preliminary treatment and primary treatment would increase capital and O&M costs by $4.9 million and $280,000/year, respectively (refer to report text for additional details). |
| *Scenario 1b*: Total construction cost, including civil works, site preparation, piping, electrical, instrumentation and control. Includes only the nitrification process stage. O&M costs do not include administration and laboratory labour.<br>Influent pump station, preliminary treatment and primary treatment would increase capital and O&M costs by $4.9 million and $280,000/year, respectively (refer to report text for additional details). |
| *Scenario 1a*: Total construction cost, including all civil works as described above. Includes only the nitrification process stage. O&M costs do not include administration and laboratory labour.<br>Influent pump station and preliminary treatment would increase capital and O&M costs by $3.9 million and $220,000/year, respectively. |
| *Scenario 3*: Total construction cost, including all civil works as described above. Includes only the nitrification process stage. O&M costs do not include administration and laboratory labour.<br>Influent pump station and preliminary treatment would increase capital and O&M costs by $740,000 and $64,000/year, respectively. |
| |
| *Scenario 1b*: Total construction cost, including all civil works for the two-stage nitrification process, including one interstage clarifier, one interstage transfer pump and effluent recirculation pumps at both stages.<br>Influent pump station, preliminary treatment and primary treatment would increase capital and O&M costs by $4.9 million and $280,000/year, respectively.<br>*Scenario 4*: Total construction cost, including all civil works for the single-stage tertiary nitrification process, including an influent pump, effluent recirculation and a final clarifier.<br>O&M costs do not include administration and laboratory labour. |
| *Scenario 1b*: Total construction cost, including all civil works for the two-stage nitrification process, including an interstage clarifier and an interstage transfer pump.<br>Influent pump station, preliminary treatment and primary treatment would increase capital and O&M costs by $4.9 million and $280,000/year, respectively.<br>*Scenario 4*: Total construction cost, including all civil works for the single-stage tertiary nitrification process, including an influent pump and a final clarifier.<br>O&M costs do not include administration and laboratory labour. |

**Table 5.6 Constructed Wetland (FWS and VSB) Capital and O&M Costs (NovaTec Consultants Inc., 1996)**

| Wastewater Flow Rates | Capital Cost ($) | | O&M Cost ($) | |
|---|---|---|---|---|
| | FWS | VSB | FWS | VSB |
| 500 $m^3$/d (0.11 MIGD) | 443 000 – 1 110 000 | 553 000 – 919 000 | 32 000 – 50 000 | 32 000 – 55 500 |
| 5000 $m^3$/d (1.1 MIGD) | 1 880 000 – 5 530 000 | 3 210 000 – 5 870 000 | 99 500 – 170 000 | 99 500 – 183 000 |

- MBBR process using free-moving plastic media from Kaldnes; and
- MBR process using immersed hollow fibre membrane technology from Zenon Environmental Inc.

Although it was assumed in each case that no new aeration tank volume would be installed, the three IFAS process options required additional secondary clarification capacity, which was incorporated into the cost estimates. It should be pointed out that in addition to providing increased biomass inventory for enhanced treatment performance and capacity, the MBR process will produce a tertiary-treated effluent quality in contrast to the other hybrid process retrofits. Furthermore, the submerged membrane system could be located in the secondary clarifier(s) if additional treatment capacity is required.

The basic influent (i.e., primary effluent) characteristics and the design conditions of the existing generic activated sludge plant design are summarized in Table 5.7. In comparison with the primary effluent characteristics of Scenario 1b described in the previous report section, the influent concentrations are lower but the average flow rate is proportionally higher, resulting in similar overall organic and TKN loading rates. This allows for rough capital cost comparisons between the new stand-alone installations that were previously described under Scenario 1b versus the various process retrofit options summarized in this section. These rough cost comparisons should be regarded with caution (WERF, 2000).

A summary of the main design bases and assumptions used for process sizing, configuration and costing included the following:

- An anoxic zone fraction of 25–35% was utilized for denitrification, depending on the process.
- The three IFAS processes include the addition of fine-bubble membrane diffusers, while about 5% of the aeration tank volume functions as a deoxygenation zone at the end of the reactor to reduce the amount of DO recycled to the anoxic zone.
- The MBBR process includes the addition of a coarse-bubble aeration system and relies on five distinct treatment zones for nitrification and denitrification.

**Table 5.7 Description of the Generic MWWTP Considered for Potential Retrofitting**

| Parameter | Value |
|---|---|
| *Influent Characteristics* | |
| Average flow rate ($m^3$/d) | 37 850 (8.3 MIGD) |
| Hydraulic peaking factor | 2.3 |
| COD (mg/L) | 225 |
| $BOD_5$ (mg/L) | 100 |
| TSS (mg/L) | 90 |
| TKN (mg/L) | 24 |
| Minimum temperature (°C) | 12 |
| *Activated Sludge Process Characteristics* | |
| Aeration basin length:width ratio | 5.4 |
| Aeration basin volume ($m^3$) | 9950 |
| Average HRT (h) | 6.5 |
| Capacity of air blowers (cfm) | 27 000 |
| Secondary clarifier surface area ($m^2$) | 1610 |

- The MBR process relies on separate air diffusers for control of the membrane process and sustenance of the biological treatment process.
- About 25% of the influent TKN was assumed to be assimilated as part of biological growth, leaving about 75% to be completely nitrified.
- The design of the three IFAS processes was based on 50% of nitrification being achieved by attached-growth biomass and the remaining 50% by suspended-growth biomass.
- The reported capital costs are based on the engineer's best estimates and communication with equipment vendors (WERF, 2000); they represent installed equipment and ancillary components but do not include the cost of other electrical, instrumentation and control, piping and civil works.

Preliminary order-of-magnitude-level capital cost estimations for the alternative retrofit processes are summarized in Table 5.8. It was emphasized in WERF (2000) that the reader is cautioned not to draw general conclusions regarding the relative cost-effectiveness of one type of retrofit process versus another. The actual cost of each system is highly project and site specific.

## Table 5.8 Capital Costs for Retrofit Upgrades for a Generic MWWTP

| Nitrogen Control Process | Total Capital Cost of Upgrade ($ Cdn) |
|---|---|
| IFAS (rope media) | 6 100 000 |
| IFAS (sponge media) | 5 830 000 |
| IFAS (plastic media) | 4 650 000 |
| MBBR (plastic media) | 7 600 000 |
| MBR (hollow fibre membranes) | 16 030 000 |

# Chapter 6

# Selection of Processes and Upgrade Alternatives

# Chapter 6

# Selection of Processes and Upgrade Alternatives

A wide range of options exists for the selection of appropriate treatment systems, upgrade alternatives and strategies for the enhanced removal of total ammonia-nitrogen from municipal wastewaters. The most appropriate process application and flow scheme will depend on numerous process- and site-specific factors. These factors and considerations related to potential greenfield applications or the upgrading of existing treatment systems would have to be prioritized and assessed through detailed engineering studies for every case. Thus, site-specific upgrade measures to provide ammonia removal according to the MWWTP's effluent objectives have to be identified and/or demonstrated on a case-by-case basis.

As a follow-up to the information on ammonia removal process alternatives that was presented in Chapter 5, with supplemental process information and considerations described in Appendix A, Chapter 6 presents a background on MWWTP upgrade alternatives. This includes information on process audits and innovative retrofit options that have shown successful applications to optimize existing processes and to minimize upgrade investments. Typical process- and site-specific factors to be taken into consideration in the preliminary selection of upgrade alternatives and optimal strategies to enhance ammonia removal from municipal wastewaters are covered. Tools such as process audits and pilot plant testing, as well as an itemization of activities that a municipal engineer will have to undertake as part of the identification, evaluation, selection, development and implementation of appropriate upgrade measures, are described. Finally, the steps involved in the solicitation and selection of a qualified consultant to carry out the identification, evaluation and selection of appropriate treatment processes and/or upgrade options are presented.

## 6.1 Background on MWWTP Processes and Upgrade Alternatives

### 6.1.1 Process Alternatives

As discussed in Chapter 5, biological processes represent the most feasible, prevalent and proven option for the removal of total ammonia-nitrogen from municipal wastewaters. Biological nitrification can readily and consistently achieve the target total ammonia-nitrogen concentration of <5 mg/L when properly designed and operated, depending on the prevailing process conditions (USEPA, 1993). A wide variety of configurations and combinations of suspended-growth and attached-growth biological processes can be applied for effective nitrification. The relative attributes of suspended-growth versus attached-growth technological options for nitrification are summarized below (USEPA, 1993; Daigger and Buttz, 1998):

- Suspended-Growth Technologies
  - more widespread application and proven experiences;
  - better understood;
  - higher degree of operational control (e.g., SRT and DO conditions);
  - enhanced process flexibility;
  - inherently more adaptable to a variety of process configurations and treatment objectives;
  - more amenable to process upgrading;
  - less influenced by other processing considerations at the MWWTP; and

- enhanced potential for low total ammonia-nitrogen discharge concentrations.

- Attached-Growth Technologies
  - enhanced stability of the relatively sensitive nitrification process;
  - often simpler to operate and maintain;
  - usually more energy efficient;
  - enhanced reliability;
  - no reliance of process on biomass settling characteristics;
  - enhanced settling characteristics of biomass (i.e., sloughed-off matter);
  - improved resistance to and recovery from shock loads and process upsets;
  - significantly lower potential for biomass washout and loss of nitrification activity; and
  - more amenable to prolonged periods of process shutdown.

In addition to these general technology-specific attributes, the critical variables for consideration in selection of the most appropriate process(es) and strategies are often site specific in nature.

### 6.1.2 Facility Upgrade Alternatives

Various approaches can be used for attaining enhanced removal of ammonia in the case of existing MWWTPs. This can involve process upgrading through optimization, retrofitting, expansion or the installation of new processes (i.e., as replacement systems or add-ons to existing processes). Cost differences between the various approaches can be considerable. The most appropriate upgrade option(s) for each case will be process and site specific. A detailed discussion of facility modifications that can be used to improve the performance of existing biological treatment processes is presented in Chapter 4 of the U.S. EPA handbook, "Retrofitting POTWs" (USEPA, 1989a). Various options for retrofitting suspended-growth and attached-growth processes are provided, a number of which are applicable to attaining enhanced nitrification. A discussion of alternatives for the retrofitting of conventional biological treatment plants to achieve biological nutrient removal is provided by Randall *et al.* (1992).

The capital cost of upgrading existing MWWTPs for nitrification through conventional process expansion (i.e., aeration tank and secondary clarification stage capacity increases) can be substantial. A 1987 total construction cost estimate for upgrading the 243 mechanical primary and secondary treatment plants in Ontario for year-round control of total ammonia to <10 mg/L was reported to be $1.72 billion (OMOE, 1990). Shortcomings of this conventional expansion approach are that the performance of existing MWWTPs can often be limited by factors that are not design related or that, through process optimization, plants can sometimes demonstrate improved performance above the recommended design capacities (USEPA, 1989a; WTC and XCG Consultants Ltd., 1992; CG&S and Hydromantis, Inc., 1996).

More innovative and lower-cost approaches to process upgrading can include process audits to determine the ultimate capabilities of existing MWWTPs, process optimization or the application of innovative retrofit technologies (WTC and XCG Consultants Ltd., 1992). In many cases, the most cost-effective approach for facility upgrading, including the capability of achieving more stringent discharge limits, is optimization of the existing processes (CG&S and Hydromantis, Inc., 1996). Even if treatment units must be added to upgrade or expand an existing facility, the magnitude of the expansion can be minimized if the existing facility is first optimized.

Capital cost savings and deferred or eliminated capital expenditure may be realized relative to conventional expansions. Other associated benefits can include improved performance, enhanced operability or reduced variability in effluent quality (WTC and XCG Consultants Ltd., 1992). Methods for the evaluation and optimization of existing MWWTPs are described in Section 6.3.2 and Chapter 7, respectively.

### 6.1.2.1 Innovative Retrofit Technologies

The application of innovative retrofit technologies for the upgrading of existing MWWTPs can represent a low-cost upgrade option relative to conventional expansion (WTC and VPISU, 1994). Some of the successfully proven, innovative retrofit techniques are listed below:

- modified conventional retrofits of activated sludge plants, which could include:
    - conversion of surface or coarse-bubble aeration to fine-pore aeration;
    - automatic DO control;
    - installation of systems for improving sludge settleability (e.g., selector zones);
    - secondary clarifier upgrades (e.g., installation of baffles and skimmers, weirs or block portion of effluent weirs relocation, improve flow distribution between the final clarifiers, installation of flocculating and energy-dissipating centre wells, increased sludge recycle capacities and/or polymer addition);
- preprecipitation for reduced wastewater BOD:TKN ratio to the nitrification process for reduced aeration requirements and enhanced process activity; and
    - conversion to step feed operations, resulting in a reduced solids loading rate to the secondary clarification stage.
- predenitrification for the recovery of alkalinity, reduced aeration requirements, increased nitrification rates and reduced potential for floating sludge in the secondary clarification stage. This would require:
    - installation of an anoxic zone (i.e., 20–40% of the total aeration tank volume) at the inlet to the aeration tank and mixed liquor recirculation; or
    - provision for intermittent aeration in the aeration tank.
- biological nutrient removal for enhanced nitrogen and phosphorus removal, reduction in chemical additions and the recovery of aeration energy:
    - development of anaerobic and anoxic zones ahead of the oxic zone in the aeration tank through the installation of baffles.
- addition of attached-growth biomass support media to the aeration basins for increased biomass inventory and year-round nitrification potential:
    - installation of media retention screens; and
    - no physical expansion of the existing plant required.

The application of innovative approaches for upgrading all 217 mechanical secondary treatment plants in Ontario was estimated to provide a potential cost savings of 67% relative to conventional expansion (WTC and VPISU, 1994). The order-of-magnitude-level overall cost estimates in 1993 for the innovative upgrade approach and for conventional expansion were \$0.6 billion and \$1.8 billion, respectively. This was based on achieving year-round control of total ammonia-nitrogen discharges to <4 mg/L. A qualitative analysis indicated that O&M costs for the innovative approaches would be similar to or lower than those associated with conventional upgrading.

It was noted that the use of innovative upgrading approaches would not, in all cases, preclude the need for expansion of certain unit processes within a facility. The applicability and cost of innovative approaches are site specific (WTC and VPISU, 1994), and the actual cost savings that can be accomplished will depend on specific circumstances at each MWWTP (WTC and XCG Consultants Ltd., 1992).

### 6.1.3 Upgrade Requirements and Options for Existing MWWTPs

Owing to the large number of existing municipal wastewater treatment facilities and because of increasingly stringent regulated discharge requirements, the upgrading of existing MWWTPs is an important issue (Daigger and Buttz, 1998). The general approach in addressing this requirement is to determine the actual capacity of the existing facility, identify and evaluate potential plant upgrade alternatives and select the most appropriate option for a particular case. All factors that affect plant capacity and capability should be considered to ensure the most accurate assessment and solution.

As discussed in Chapter 3, various levels of wastewater treatment and process types are employed at existing MWWTPs in Canada (Table 3.1). Process upgrade requirements and technical alternatives to enhance ammonia removal depend on the type, design and limitations of existing plant processes, their inherent flexibility for optimization, reconfiguration and/or retrofitting and the available treatment capacity and capabilities of the existing plant.

The three basic types of existing on-site wastewater treatment are primary treatment plants, lagoon-based secondary treatment plants and higher-rate biological secondary or tertiary treatment systems utilizing either suspended- or attached-growth technologies. Described below are possible upgrade requirements and some potential remedial options for the enhanced control of effluent total ammonia-nitrogen discharges for each of these three categories of existing treatment facilities.

#### 6.1.3.1 Upgrading Existing Primary Treatment Plants

For existing primary treatment plants, a secondary treatment process with year-round nitrification capability would need to be installed for total ammonia-nitrogen control. Selection of the most appropriate process depends on various factors as defined below. Furthermore, consideration would have to be given to process-specific advantages and limitations (Appendix A), including associated process costs.

Chemically assisted primary treatment plants can facilitate the design and operational performance of a new nitrifying secondary treatment process due to the reduced cBOD:TKN ratio in the effluent. The higher resultant specific nitrification rates will reduce secondary treatment process size and cost and should enhance performance characteristics. Conversely, depending on the chemical dosage utilized, high amounts of alkalinity may be consumed and may require supplementation to enhance nitrification activity.

#### 6.1.3.2 Upgrading Existing Lagoon-Based Secondary Treatment Plants

As described in Chapters 3 and 5, many of the numerous lagoon-based secondary treatment plants that are utilized in Canada (i.e., continuous-discharge aerated and aerated facultative lagoons and conventional seasonal-discharge facultative lagoons) would require upgrading to achieve effluent discharge total ammonia-nitrogen concentrations of less than 5 mg/L on a consistent basis. This is due to negative effects of cold-climate temperature conditions on the low-rate processes during the wintertime and the possible resolubilization and release of ammonia during the springtime resulting from the decomposition of accumulated sediment.

As discussed in Chapter 5 and elaborated upon in Appendix A, potential lagoon process upgrade options for consistently achieving the target effluent discharge total ammonia-nitrogen concentration of <5 mg/L through nitrification include:

- installation of adequate long-term facultative storage capacity to permit controlled annual discharge of well-treated lagoon effluent during autumn;
- installation of a new add-on posttreatment stage downstream of the existing lagoon for control of total ammonia-nitrogen to target levels in continuous- or seasonal-discharge applications;
- installation of a new extended aeration activated sludge process for simultaneous carbon oxidation and nitrification upstream of an existing continuous-discharge lagoon process;
- conversion of the existing lagoon volume into an extended aeration activated sludge process followed by a polishing lagoon zone; and
- retrofitting of the existing lagoon to include a segregated zone(s) with attached-growth support media and aeration for the development and retention of nitrifying process biomass for year-round control of total ammonia-nitrogen to target levels in continuous-discharge applications.

In cases where there is limited land availability and/or where wastewater flow rates are high, the add-on posttreatment process option likely represents the most viable and compact upgrade approach. Although considered promising, the alternative involving retrofitting of lagoon zones with attached-growth support media is considered novel, with no known full-scale applications and experiences to date. Elaboration on some of the different lagoon process upgrade options is provided below.

#### 6.1.3.2.1 Installation of Add-On Nitrifying Posttreatment Stage

Posttreatment of aerated lagoon or facultative lagoon effluent could employ any of the alternative attached-growth nitrification processes described in Section 5.1.4. Additional technical details on the alternative attached-growth nitrification processes and considerations are provided in Appendix A.

*Low-flow applications*

The most suitable add-on posttreatment technologies for small rural communities would be those that are simple in design and operation, such as constructed wetlands (NovaTec Consultants Inc., 1996; USEPA, 2000), intermittent sand filters or recirculating sand filters, owing to their relative ease of operation, low capital and operating cost and ability to produce a high-quality effluent (USEPA, 1992, 1993; OMOE and Environment Canada, 1993; McMaster University and WTI, 1996). Of these three technologies, the recirculating sand filter technology would likely offer the greatest potential for year-round ammonia removal in cold-climate continuous-discharge applications. The intermittent sand filter technology should be considered for use only as a posttreatment stage in seasonal-discharge applications. As described in Section 5.1.4, a successful pilot-scale demonstration study revealed the relative advantage of the recirculating sand filter over the intermittent sand filter technology during wintertime cold-temperature operation (Rushbrook and Urban, 1998). Thermal analysis of constructed wetlands could be used to determine the biological and physical stability of the treatment in cold-climate operation. At low temperatures, especially lower than 1°C, a severe reduction or failure of the nitrification process may occur with FWS and VSB systems. In such cases, seasonal wetland posttreatment may be contemplated for seasonal-discharge applications only.

Although the intermittent sand filter technology has been recommended as a suitable option for upgrading effluent quality from seasonal-discharge lagoon systems in the province of Ontario, where the integrated system is referred to as the New Hamburg Process (OMOE and Environment Canada, 1993), caution was made of performance uncertainties for extreme cold climate applications. Intermittent sand filter polishing is reportedly capable of generating a very high-quality effluent during the spring and autumn discharge periods, including residual total ammonia-nitrogen levels of less than 4 mg/L. However, intermittent sand filter polishing is not suitable for year-round continuous-discharge applications, as marginal benefits in total ammonia removal have been found during the coldest winter months based on U.S. experiences (OMOE and Environment Canada, 1993). Furthermore, the intermittent sand filter process would typically be taken off-line during winter operations in cold-climate locations owing to freezing problems, necessitating wintertime storage of lagoon effluent. Although the intermittent sand filter process has been demonstrated to rapidly recover nitrifying activity following prolonged wintertime shutdowns, the ability to achieve effective ammonia control upon resumption of operations in cold-climate locations is uncertain, as there are limited reported full-scale experiences (OMOE and Environment Canada, 1993).

### *Advanced process applications*

More advanced and complex attached-growth processes such as the trickling filter, RBC or BAF technologies could be employed for larger, continuous-flow posttreatment applications (Daigger and Buttz, 1998) and would offer an increased degree of process control. For example, a trickling filter is used for posttreatment of lagoon effluent by the innovative Pond Enhanced Treatment and Operation process described by Daigger and Buttz (1998). This energy-efficient and mechanically simple process is utilized both for the removal of algae from the effluent of an anaerobic plus facultative lagoon process and for nitrification. Application of the RBC or the BAF process for lagoon effluent polishing is an effective upgrade option, although use of the latter technology would represent a relatively complex polishing alternative, including a high degree of instrumentation and control and the requirement for increased operator skills, but potentially producing a superior-quality effluent.

### *Add-on process considerations*

In all of the cases noted above, the existing lagoon process would provide equalization capacity to enhance the stability of a downstream add-on nitrification process, thus resulting in an enhanced consistency in effluent discharge quality. The upstream lagoon process can remove a large portion of the influent BOD and TSS and will concomitantly reduce the $\text{cBOD}_5/\text{TKN}$ ratio to a more favourable level for nitrification in the posttreatment stage.

A consideration and limiting factor in the use of an add-on tertiary nitrification stage would be the low wintertime temperatures (e.g., 5°C or less) that can occur in lagoon processes and effluent discharges in cold-climate applications (Environment Canada, 1987). Low temperature could have a negative effect on the downstream nitrification process. Low specific nitrification rates could be expected, requiring large biomass inventories. Although it is theoretically possible to maintain nitrification activity at temperatures as low as 2°C, as has been reported for attached-growth processes, as described in Chapter 5, limited experiences and information exist for full-scale applications of such processes operating below 5°C. Accordingly, field pilot plant investigations are strongly recommended for prospective biological nitrification process applications at conditions of less than 10°C to assess viability and to determine appropriate design conditions on a case-by-case basis. The posttreatment nitrification process should be started up well before the onset of cold weather conditions to facilitate the development and establishment of active nitrifying biomass.

Another process consideration in the application of tertiary nitrification is the relatively low ammonium concentrations in lagoon effluent discharges during the summertime due to increased up-front ammonia removal. This would significantly reduce the total ammonia-nitrogen loading to the posttreatment stage, which could reduce nitrification activity. Such conditions should be accommodated by the nitrifying biomass through a reduction in specific activity. A possible option to alleviate this situation would be to divert a portion of the raw sewage directly to the nitrification process stage during these warm weather periods to increase influent total ammonia-nitrogen loadings. However, elevated influent suspended solids concentrations and an increased cBOD:TKN ratio may be concerns for nitrification efficiency and certain types of attached-growth processes. Alternatively, the downstream nitrifying processes could be taken off-line and idled during the summer season. This would necessitate subsequent startup and reacclimation of the process during autumn prior to the onset of colder temperature conditions, to ensure proper performance during winter and springtime.

For cases involving existing multicell mechanically aerated lagoons or aerated facultative lagoons, the new nitrification process could conceivably be installed to treat the effluent from the initial aerated cell(s) and to subsequently discharge nitrified effluent into the other downstream aerated or facultative lagoon cells. This process arrangement would result in warmer wintertime effluent temperatures to the nitrifying process, adequate total ammonia-nitrogen loads and an acceptable effluent cBOD:TKN ratio to the nitrification process in the summertime. Possible uncontrolled seasonal releases of ammonia may result from the decomposition of sludge sediment in the downstream facultative lagoon cell, which could increase total ammonia-nitrogen concentrations in the final effluent discharge, particularly during warm temperature conditions. This could be alleviated by dredging accumulated sludge deposits every few years, depending on the size of the lagoon (OMOE and Environment Canada, 1993).

#### 6.1.3.2.2 Installation of an Upstream Nitrification Process

Installation of a mechanical secondary treatment process for simultaneous carbon oxidation and nitrification upstream of an existing continuous-discharge lagoon system could represent another approach for providing year-round nitrification. However, this would increase overall system size, complexity and capital cost, as well as O&M requirements and costs. This configuration would result in warmer wastewater temperatures to the nitrification process; however, the cBOD:TKN ratio would be higher than for the tertiary treatment application described in the previous subsection. Consequently, the size of the nitrification would be relatively large. The existing lagoon would serve as an effluent polishing stage or could be bypassed.

*Sutton Process*

An example of such a two-stage process configuration is the Sutton Process, which has been applied at a number of MWWTPs in Ontario. It consists of a single-stage nitrifying extended aeration activated sludge process followed by a facultative lagoon for effluent polishing. WAS is discharged on a controlled basis to the downstream polishing lagoon to minimize sludge handling and disposal requirements and associated costs. A schematic of the Sutton Process is shown in Figure 6.1, as taken from OMOE and Environment Canada (1993).

The Sutton Process applications in Ontario have mainly involved retrofits to existing lagoons, where a portion of the lagoon volume is converted to an extended aeration activated sludge process, with the remaining lagoon volume used for polishing of the secondary clarified effluent. The upstream extended aeration activated sludge process stage is designed to achieve virtually complete year-round nitrification (i.e., <1 mg total ammonia-nitrogen/L).

An example of a conversion of a full-scale aerated facultative lagoon into a Sutton Process was the Lindsay, Ontario, MWWTP, which has a design capacity of 16 MLD (OMOE and Environment Canada, 1993). This involved the conversion of the two aerated lagoon cells into an extended aeration activated sludge process, with secondary clarified effluent directed to the existing facultative lagoons for polishing. An example of a new greenfield application of the Sutton Process is the 825 $m^3/d$ MWWTP installation in Cookstown, Ontario.

Unfortunately, over time, sludge stored in the lagoon undergoes cell lysis releasing nutrients back into the water. This degrades the quality of the lagoon contents, in some cases to that below the activated sludge plant effluent (same as the lagoon influent). As a result of these findings, this process is no longer used in its original form, and any installations set up in Ontario have been modified to eliminate this problem. (Private conversation with R. (Mano) Manoharan, Ontario Ministry of the Environment, January 2003.)

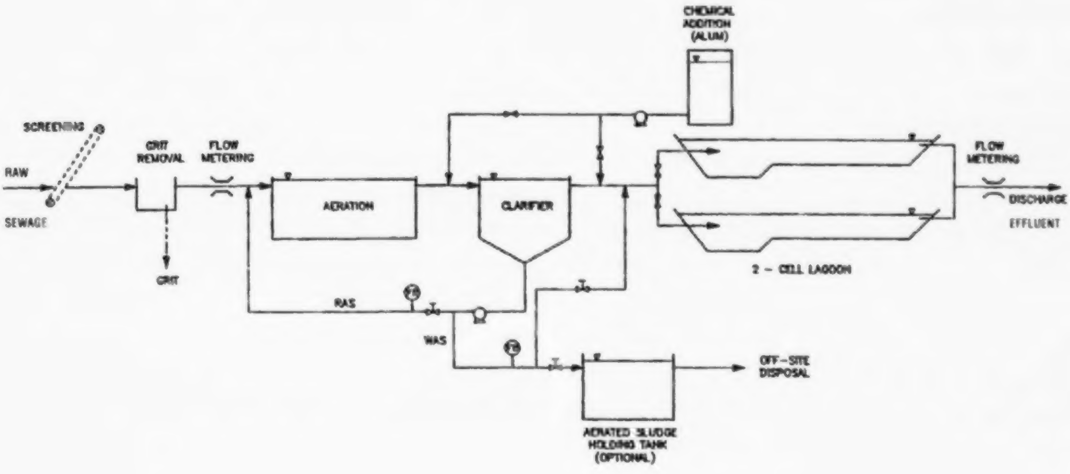
*Low-flow applications*

In low-flow applications in small communities, a simpler option could involve the installation of an extended aeration activated sludge package plant upstream of an existing lagoon process. The package plant could conceivably be operated without the controlled wasting of excess sludge. By allowing excess solids to overflow in the package plant effluent, O&M requirements and costs would be reduced. In such cases, the downstream lagoon process would be relied upon to accommodate the solids overflow from the activated sludge process. Sludge accumulation in the lagoon would have to be properly managed.

#### 6.1.3.2.3 Lagoon Retrofitting

The addition of attached-growth support media to a segregated portion of an existing lagoon system may represent a cost-effective upgrade option. This retrofit would likely require the installation of curtains or walls for partitioning of the lagoon into different zones. The zone containing the attached-growth support medium would require the installation of media retention screens and submerged aerators for the required aeration and mixing. However, there would be no need for effluent pumping, unlike the alternative case of an add-on nitrification process. This lagoon retrofit approach would require pilot-scale testing and/or full-scale demonstration owing to the absence of reported full-scale experiences and information.

Figure 6.1 Schematic of the Sutton Process


CHEMICAL ADDITION (ALUM)
SCREENING
RAW
SEWAGE
GRIT REMOVAL
FLOW METERING
GRIT
AERATION
CLARIFIER
FLOW METERING
DISCHARGE
EFFLUENT
2 - CELL LAGOON
RAS
WAS
OFF-SITE DISPOSAL
AERATED SLUDGE HOLDING TANK (OPTIONAL)

#### 6.1.3.2.4 Installation of Add-On Physical/Chemical Posttreatment

Although biological processes are the preferred option for total ammonia-nitrogen control, physical/chemical polishing of lagoon effluent represents a possible upgrade alternative. Physical/chemical posttreatment could be applied on an as-required basis (e.g., seasonally), with short startup times, and taken off-line when not required. Some of the available physical/chemical processes would be insensitive to the low temperatures of lagoon effluents in the wintertime. However, as described in Section 5.2 and in Appendix A, there are limitations to these processes, and caution must be exercised in deciding upon their application (USEPA, 1993).

Breakpoint chlorination is one option for occasional polishing of lagoon effluent. This is practised at the Niagara-on-the-Lake, Ontario, MWWTP during cold weather periods when the aerated facultative lagoon process is unable to control total ammonia-nitrogen to target levels. However, breakpoint chlorination is difficult to control. Furthermore, undesirable by-products may be generated, and effluent dechlorination would be required.

Ion exchange technology could be employed as a posttreatment option, as it is applicable under cold temperature conditions (USEPA, 1971). This costly process option would require pretreatment using effluent filtration to prevent column plugging. The ion exchange medium (e.g., clinoptilolite) is expensive, and the treatment or disposal of spent regenerant solution would be required.

Air stripping and reverse osmosis technologies are generally not considered to be feasible options for lagoon effluent posttreatment for total ammonia-nitrogen control. Air stripping is not applicable for wastewater temperatures of below about 10°C, and the cost of reverse osmosis technology would be high relative to the other options. Reverse osmosis would require up-front suspended solids removal, and filtrate flux levels are reduced by low temperatures.

### 6.1.3.3 Upgrading Existing High-Rate Mechanical Secondary Treatment Plants

Numerous options are available for the upgrading of existing medium- to high-rate mechanical secondary treatment plants that either do not nitrify or achieve only partial and/or seasonal nitrification. Many of these upgrade options are described in detail by Randall *et al.* (1992) and Daigger and Buttz (1998). The suitability of options will depend on process- and site-specific factors, as described below.

Nitrification requirements affect the following areas of the existing plant:

- aeration tank volume and configuration;
- oxygen transfer system capacity;
- secondary clarification stage capacity (if applicable); and
- process operations and control.

Depending on the characteristics of the existing mechanical plant and situation, a wide range of applicable upgrade requirements can exist. CG&S and Hydromantis, Inc. (1996) and Daigger and Buttz (1998) described comprehensive approaches for determining the capacity and capability of existing wastewater treatment systems, as described in Section 6.3.2. Based on such assessments, the most appropriate upgrade options can be identified, evaluated and selected. The most cost-effective plant upgrade will result in the maximum use being made of the existing facilities (Daigger and Buttz, 1998).

Existing plants that already experience partial and/or seasonal nitrification capabilities may be able to achieve more complete and consistent nitrification to target discharge levels through optimization of process operations (e.g., increase in SRT, enhanced biofilm control, adjustments to operating DO and pH levels, etc.), retrofitting (e.g., upgrading to a more efficient aeration or ventilation system, installation of membranes into aeration tanks and/or clarifier tanks for improved biomass retention, etc.) and/or reconfiguration (i.e., installation of baffles for compartmentalization and/or flow rearrangement to enhance overall nutrient control).

Existing secondary treatment plants that do not nitrify may require increased and more costly upgrade measures involving expansion (e.g., aeration tank volume and/or secondary clarification capacity increases), retrofitting (e.g., aeration system upgrades for improved oxygen transfer efficiency and increased oxygen transfer rates, addition of supplemental attached-growth support media into suspended-growth processes, addition of filtration membranes into aeration tanks or clarifiers for biomass retention, increased instrumentation and control, etc.) or the installation of a separate downstream nitrification stage.

Installation of a new add-on posttreatment stage could involve either a nitrifying suspended-growth activated sludge process or any of the various nitrifying attached-growth processes described in Chapter 5, depending on the flow rate. The addition of a trickling filter, BAF or RBC process stage to upgrade an existing mechanical secondary treatment plant has proved to be an effective approach to accomplish nitrification (Daigger and Buttz, 1998). For example, the Guelph, Ontario, MWWTP previously installed a nitrifying RBC process stage downstream of an existing activated sludge plant to ensure consistent year-round control of total ammonia-nitrogen to concentrations of less than 5 mg/L.

Add-on process implementation requires major construction, often including a dedicated new wastewater pumping facility. It involves the assistance of engineering consultants and sometimes proprietary systems suppliers (Daigger and Buttz, 1998). Retrofit options depend on the type, design and capacity of the existing process(es), which will have to be assessed on a case-specific basis.

## 6.2 Preliminary Selection of Facility Upgrade Measures

The selection of MWWTP upgrade measures for enhanced nitrification performance involves the identification of specific requirements for an existing treatment facility, with consideration given to process-specific and site-specific factors. In Section 6.2.1, two screening matrices are presented to assist with the preliminary identification and selection of appropriate upgrade measures. The first matrix outlines a number of upgrade categories and various associated upgrade measures. The second matrix identifies those measures listed in the first matrix that are applicable for a given upgrade category, depending on the level of treatment employed at an existing site (i.e., a site possessing either no treatment facilities or utilizing a non-nitrifying or only partially nitrifying plant that is required to achieve improved performance).

In Section 6.2.2, various process-specific and site-specific factors for consideration related to the application of nitrification systems are identified and discussed. Based on information such as wastewater characterization data, site constraints and the advantages and limitations of different treatment process types (i.e., as described in Chapter 5 and Appendix A), the applicable upgrade alternatives and measures identified from the matrices can be properly screened, assessed and prioritized, leading to the selection of the most appropriate upgrade measure(s) for implementation. This would need to be evaluated on a case-by-case basis.

The screening matrices together with the other supporting information contained in this chapter can provide a useful preliminary indication of applicable solutions for various treatment scenarios, which can involve small-, medium- or large-flow applications requiring enhanced ammonia removal.

### *6.2.1 Upgrade Alternatives for Enhanced Ammonia Removal*

The identification and preliminary selection of upgrade alternatives for enhanced ammonia removal from municipal wastewaters are presented in this section. This is based on a range of possible measures that are defined under five specific upgrade categories, namely:

- ***New installation:*** As a greenfield application, a replacement facility or a process installed in parallel to an existing plant;
- ***Optimization:*** Operational modifications to existing plant process(es);
- ***Retrofit:*** Physical modifications to existing plant process(es);
- ***Expansion:*** Volume increase to existing treatment system unit(s); and
- ***Add-on nitrifying process:*** Addition of a separate nitrifying process stage in series (i.e., either upstream or downstream) with liquid train process stages at an existing MWWTP.

These five upgrade categories and various associated measures are outlined in Table 6.1, which is the first of the two screening matrices. This first matrix assigns a specific numeric value to each listed measure, under each of the five respective upgrade categories, for identification purposes.

The category of "new installation" (i.e., first category shown in Tables 6.1 and 6.2) can apply in cases where there is no existing treatment plant or where the decommissioning and replacement of an existing plant are planned. Therefore, the only specific measure listed under this category is for the installation of a nitrifying process (i.e., any one of the three general types that are indicated).

The other four upgrade categories pertain to the modification of existing facilities and, therefore, have a greater number of possible measures associated with them. Depending on the case, a single measure may be sufficient, or, alternatively, a number of measures could be implemented in combination as part of facility upgrading. Many of the listed measures are described in the preceding Sections 6.1.2 and 6.1.3, while others are discussed later in Chapter 6 and in Chapters 7 and 8.

The second matrix table (Table 6.2) outlines various applicable upgrade alternatives depending on the treatment scenario (i.e., level of treatment existing at a candidate municipal site). The specific applicable upgrade measures, according to each of the five categories, are identified by cross-referencing with Table 6.1 using the assigned numeric values.

The information from the two matrices serves as a preliminary screening tool for identifying a range of potential upgrade measures to achieve enhanced ammonia removal for candidate sites, depending on the existing level of treatment installed, if any. For each of the treatment scenarios listed in the first column in Table 6.2 (i.e., the existing level of on-site treatment and/or treatment process utilized), it is assumed that consistent year-round total ammonia-nitrogen removal to less than 5 mg/L cannot be achieved, thus the requirement for upgrading. Either the candidate site possesses no treatment or else the existing level of treatment may be limited to primary treatment or non-nitrifying secondary treatment. Each of the applicable upgrade measures identified for the different cases is considered to have the potential to consistently achieve effluent total ammonia-nitrogen concentrations of less than 5 mg/L, including cold-climate applications.

A case study of a full-scale treatment plant upgrade project is summarized in Section 6.2.1.1 to illustrate the use of the two screening matrices.

Selection of the most appropriate upgrade alternatives for a given application may be obtained through process audits (Section 6.3.2.1) at an existing plant and/or pilot plant testing (Section 6.3.2.2). MWWTP audits may help to identify cost-effective upgrade alternatives such as optimization and/or retrofitting that can solve process bottlenecks and enhance ammonia removal. Process optimization is described in Chapter 7.

The selection of upgrade categories and applicable measures should be approached with a progressive step-wise point of view. For instance, optimization of existing on-site processes should be conducted prior to the physical retrofitting and expansion of the system or the installation of add-on processes. The best approach for a particular upgrade application depends on the design and limitations of the existing treatment system and other site-specific considerations. Accordingly, the screening matrices for preliminary identification and selection of potential upgrade alternatives and measures should be used with consideration given to process- and site-specific factors, as discussed in

Section 6.2.2. These factors should be verified according to individual MWWTP circumstances.

#### 6.2.1.1 Example of Screening Matrices Use

A case study of a full-scale treatment plant upgrade project is summarized below to illustrate the use of the two screening matrices. Various aspects of plant assessment, process optimization and physical retrofitting were utilized for cost-effectively achieving year-round nitrification and demonstrating increased treatment capacity at the City of Windsor Little River MWWTP. A more comprehensive case study description of this plant and upgrade project is presented in Section 9.1.1.

The Little River MWWTP is a secondary treatment facility with a nominal capacity of 64 MLD, divided into two plants. Plant No. 1 was not designed for nitrification, while Plant No. 2 was installed as part of a facility expansion in 1989 and was designed according to the Ontario Ministry of the Environment guidelines, including 6 hours average HRT in the aeration system. Comprehensive process audits were conducted at both plants using numerous techniques to assess actual treatment capacity and identify suitable process optimization opportunities. Based on the findings of the process audits, a number of upgrade measures were implemented for improved treatment performance, including consistent nitrification and a plant rerating to 73 MLD, in contrast to an originally proposed facility derating to 46 MLD for nitrification (Newbigging *et al.*, 1994; CG&S and Hydromantis, Inc., 1996).

A summary of the upgrade measures and associated categories that were implemented at the Little River MWWTP is given below. The applicable treatment scenario for this particular case study example was an existing conventional activated sludge process (Plant No. 1) at the site (i.e., sixth scenario listed in Table 6.2).

##### 6.2.1.1.1 New Installation

A suspended-growth nitrification process (i.e., Plant No. 2) was installed in parallel with the original Plant No. 1, for a 27 MLD increase in treatment capacity and enhanced treatment performance. [This corresponds to upgrade ***Measure #1*** under the category of "New Installation" as described in Table 6.1 and shown in Table 6.2.]

##### 6.2.1.1.2 Optimization

Based on the findings of the detailed process audits conducted at the two plants [corresponding to upgrade ***Measure #1*** under the "Optimization" category in the two matrix tables], the following optimization measures were implemented:

- Modelling and simulation were applied, including the development of a sludge wasting model for enhanced SRT control [corresponds to upgrade ***Measure #3*** under the "Optimization" category];
- On-line monitoring (i.e., temperature and DO concentration in the aeration tanks, flow rates and suspended solids concentrations for the RAS, WAS and effluent streams) was used for the audit work, and a Supervisory Control and Data Acquisition (SCADA) system was installed for continuous process monitoring and control [corresponds to upgrade ***Measure #6***];
- On-line process control was implemented, including DO control and SRT control through a sludge wasting control strategy via the SCADA system [corresponds to upgrade ***Measure #7***];
- Chemical addition to control sludge bulking [corresponds to ***Measure #9***];
- Step feed operation in Plant No. 2 [corresponds to ***Measure #10***];
- Enhanced process operation and control (e.g., SRT and DO control, air diffuser cleaning

strategy, operator training initiatives, etc.) [corresponds to ***Measures #11, #12, #20, #21***].

#### 6.2.1.1.3 Retrofit

Based on process audit findings, the following process retrofits were made:

- Installation of on-line instrumentation and a SCADA system [corresponds to upgrade ***Measure #1*** under the "Retrofit" category].
- The plant utilizes chemically assisted primary treatment [corresponds to ***Measure #4***].
- An anoxic selector zone was installed [corresponds to ***Measure #8***].
- Replacement of mechanical aerators with higher-efficiency fine-pore diffusers in the aeration tank of Plant No. 1 [corresponds to ***Measure #10***].
- Baffles were installed into the secondary clarifiers of Plant No. 1 [corresponds to ***Measure #27***].

No upgrade measures listed under the "Expansion" and "Add-On Process" categories were implemented as part of the actual facility upgrading.

In summary, Table 6.2 reveals 50 upgrade measures that may be applicable for the treatment scenario involving the required upgrading of an existing conventional activated sludge plant. Measures associated with each of the five upgrade categories represent possible options for achieving consistent control of total ammonia-nitrogen discharge concentrations to below 5 mg/L. Not all measures listed in Table 6.1 apply to this particular treatment scenario, as some measures are relevant only to attached-growth or lagoon processes.

Based on site-specific considerations and assessments in the case of the Little River MWWTP, 16 of the 50 applicable measures, from three of the five upgrade categories shown in Table 6.2, were actually implemented. These upgrade measures have demonstrated the potential to cost-effectively achieve consistent nitrification at greater than design loading rates, as described in Section 9.1.1.

### 6.2.2 Factors and Considerations

Further to the preliminary information contained in the screening matrices (i.e., Tables 6.1 and 6.2), process selection considerations are generally influenced by a number of process-specific and site-specific factors. As a minimum, the following factors should be given consideration in process selection (WEF and ASCE, 1992):

- effluent discharge criteria (e.g., allowable total ammonia-nitrogen discharge concentration);
- raw wastewater characteristics;
- site constraints;
- process capabilities;
- compatibility of the candidate process with existing processes and overall treatment system flow sheet;
- process control;
- O&M factors and requirements;
- sidestreams and recycle streams;
- solids production;
- air emissions;
- energy requirements;
- space requirements;
- occupational health and safety aspects; and
- cost.

The relative weight or importance assigned to the applicable factors and considerations for process selection are project specific.

## Table 6.1 Upgrade Measures for Enhanced Ammonia Removal from Municipal Wastewaters — Matrix 1

| Upgrade Measure | Upgrade Categories | | | | |
|---|---|---|---|---|---|
| | **New Installation** | **Optimization** | **Retrofit** | **Expansion** | **Add-On Process** |
| 1 | Suspended growth nitrification [single & two stages] | Plant audit | Instrumentation and control (I&C) | Primary clarifier | Seasonal storage pond |
| 2 | Attached-growth nitrification [single & two stages] | CCP[1] | Equalization of influent and/or return streams | Aeration tank [suspended- and attached-growth processes] | Physical/chemical processes |
| 3 | Combined suspended- and attached-growth process [two-stage nitrification] | Modelling & simulation | Fine screens | Modular process additions [SBR, RBC, BAF, MBR] | All organic removal suspended-growth processes |
| 4 | | Pilot plant testing | Chemically assisted primary treatment | Aeration system capacity | All nitrifying suspended-growth processes |
| 5 | | Debottleneck (liquid and solids trains) | Alkalinity supplementation | Attached-growth media surface area | Extended aeration activated sludge (including package plants for low-flow applications) |
| 6 | | On-line monitoring | Process/flow pattern reconfiguration | Ventilation system capacity [trickling filter] | All organic removal attached-growth processes |
| 7 | | On-line process control | Step feed configuration | Secondary clarifier | All nitrifying attached-growth processes |
| 8 | | Return streams management (recycle loads) | Selector zone | Biosolids processing stages | Constructed wetlands (low-flow applications) |
| 9 | | Chemical addition (BOD reduction, detoxification, improved sludge settleability, etc.) | Pre- or postdenitrification (anoxic zone/intermittent aeration) | | Recirculating sand filters (low-flow applications) |
| 10 | | Step feed operation | Aeration system enhancement (increased oxygen transfer rates and efficiency) | | Intermittent sand filters (low-flow applications) |
| 11 | | Enhanced nitrification process operations and control (i.e., SRT, biofilm excess growth, DO, pH, alkalinity, temperature) | Mixing system enhancement (e.g., by aeration, recirculation, agitation, rotational speed) | | |
| 12 | | Aeration control | Insulation (for reduced heat losses) | | |
| 13 | | On/off aeration | Baffles in aeration tanks (for segregated treatment zones) | | |
| 14 | | Cycle times [SBR or on/off aeration processes] | Pure oxygen supplementation | | |
| 15 | | SK (dosing intensity) set points, including distributor speed control [trickling filter] | Membranes for sludge retention | | |
| 16 | | Treated effluent recycle rates [trickling filter wetting efficiency and BAF filtration velocity] | Lagoon process conversion to extended aeration activated sludge plus polishing [e.g., Sutton Process] | | |
| 17 | | Shaft rotation control [RBC and SBC] | Attached-growth support media replacement | | |
| 18 | | Backwash frequency [BAF] | Static and hydrostatic distributor conversion to variable-speed rotating distributor [trickling filter] | | |
| 19 | | Sludge recycle rates | Ventilation system capacity increase [trickling filter] | | |
| 20 | | Sludge management | Provision for effluent recirculation [trickling filter, BAF] | | |
| 21 | | Sludge inventory | Baffles to divide passes [RBC and SBC] | | |
| 22 | | Periodic reversal of influent flow to nitrifying stages [RBC or SBC] | Wastewater distribution uniformity [RBC, BAF] | | |
| 23 | | | Provision to alternate feed point to nitrifying stages (i.e., reverse flow) [RBC] | | |
| 24 | | | Wastewater peak flows redistribution; from series to parallel reactor arrangement [BAF] | | |
| 25 | | | Attached-growth support media addition to activated sludge aeration basins or lagoons [hybrid system] | | |
| 26 | | | Screens for free-moving attached-growth media [hybrid systems] | | |
| 27 | | | Baffles in clarifiers (for improved performance) | | |
| 28 | | | Secondary clarifier polymer addition | | |
| 29 | | | Sludge recycling capacities | | |
| 30 | | | Sludge wasting capacities | | |
| 31 | | | Variable-speed pumps or control valves for RAS and WAS rate control | | |
| 32 | | | Sludge processing upgrades | | |

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[1] CCP = Composite Correction Program.

## Table 6.2 Applicable Upgrade Alternatives for Enhanced Ammonia Removal for Various Treatment Scenarios — Matrix 2

| Existing On-Site Treatment System | MWWTP Upgrade Alternatives[1] | | | | | |
|---|---|---|---|---|---|---|
| | | | | | Add-On Process | |
| | New Installation | Optimization | Retrofit | Expansion | Upstream | Downstream |
| None | 1, 2, 3 | | | | | |
| Primary treatment | 1, 2, 3 | | | | | |
| Chemical-assisted primary treatment | 1, 2, 3 | | | | | |
| Facultative lagoon[2] | 1, 2, 3 | 20[3] | 1, 5, 10, 11, 13, 16, 25, 26 | | 5 | 1, 2, 4, 7, 8, 9, 10 |
| Aerated lagoon | 1, 2, 3 | | 1, 5, 10, 11, 12, 13, 14, 16, 25, 26 | | 5 | 2, 4, 7, 8, 9 |
| Conventional activated sludge | 1, 2, 3 | 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 19, 20, 21 | 1, 2, 4, 5, 6, 7, 8, 9, 10, 11, 13, 14, 15, 25, 26, 27, 28, 29, 30, 31, 32 | 1, 2, 4, 7, 8 | 3, 6 | 2, 4, 7 |
| High-rate activated sludge (including pure oxygen activated sludge) | 1, 2, 3 | 1, 2, 3, 4, 5, 6, 7, 8, 9, 11, 12, 19, 20, 21 | 1, 2, 4, 5, 6, 8, 9, 10, 11, 13, 14, 25, 26, 27, 28, 29, 30, 31, 32 | 1, 2, 4, 7, 8 | | 2, 4, 7 |
| Extended aeration activated sludge | 1, 2, 3 | 1, 2, 3, 5, 6, 7, 8, 11, 19, 20, 21 | 1, 2, 5, 6, 8, 9, 12, 13, 15, 25, 26, 27, 28, 29, 30, 31, 32 | 2, 7, 8 | | 2, 4, 7 |
| Sequencing batch reactor | 1, 2, 3 | 1, 2, 3, 4, 5, 6, 7, 8, 9, 11, 12, 13, 14, 20, 21 | 1, 2, 4, 5, 6, 9, 10, 11, 14, 15, 30, 31, 32 | 1, 3, 4, 8 | 3, 6 | 2, 4, 7 |
| Trickling filter | 1, 2, 3 | 1, 2, 3, 4, 5, 6, 7, 8, 9, 11, 12, 15, 16, 20, 21 | 1, 2, 4, 5, 6, 12, 17, 18, 19, 20, 27, 28, 30, 32 | 1, 2, 5, 6, 7, 8 | 3, 6 | 2, 4, 7 |
| Rotating biological contactor | 1, 2, 3 | 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 17, 20, 21, 22 | 1, 2, 4, 5, 6, 7, 10, 11, 12, 17, 21, 22, 23, 27, 28, 30, 32 | 1, 3, 4, 5, 7, 8 | 3, 6 | 2, 4, 7 |
| Aerated submerged fixed-bed reactor (BAF) | 1, 2, 3 | 1, 2, 3, 4, 5, 6, 7, 8, 9, 11, 12, 16, 18, 20, 21 | 1, 2, 3, 4, 5, 6, 9, 10, 17, 20, 22, 24, 30, 32 | 1, 3, 4, 5, 8 | 3, 6 | 2, 4, 7 |

[1] Specific applicable measures that are enumerated under each alternative upgrade category and treatment scenario are defined and described in Table 6.1.

[2] Continuous-, seasonal- or annual-discharge systems.

[3] Optimization measure that applies to annual-discharge lagoons.

#### 6.2.2.1 Process-Specific Factors

Selection of the upgrade technologies and approaches depends on factors affecting the ability of the process to achieve nitrification. These can include process-specific factors within the following areas: hydraulics, primary treatment, secondary treatment, secondary clarification, sidestream management, biosolids management and operational considerations for the new process or the existing process to be upgraded. The following items summarize and discuss the major process-related factors for enhancing ammonia removal in an existing or a new MWWTP. Process-limiting factors can include insufficient oxygen supply rate and insufficient aeration tank HRT, poor sludge settling characteristics and low clarification stage capacity (WTC and VPISU, 1994).

Some of these process-specific factors are mentioned as direct upgrade measures in the selection matrices in Tables 6.1 and 6.2, while others are process-specific factors affecting the upgrade measures.

**General**

*Factors applying to all ammonia removal processes and upgrade alternatives identification approach*

- Regulated effluent discharge limits (e.g., total ammonia-nitrogen);
- Raw wastewater characteristics;
- Upgrading of existing on-site treatment system depends on flexibility of process operational optimization and process retrofit and reconfiguration possibilities. The capacity and geometry of the aeration and secondary clarification stages are factors for enhanced ammonia removal implementation with an existing suspended- or attached-growth biological treatment system;
- General arrangement, design, geometry and capacity of existing treatment plant;
- Treatment performance potential of ammonia removal processes;
- Impacts of upgrade alternatives on ammonia removal efficiency;
- Modular characteristics of prospective new processes in view of possible future plant expansion;
- Number and location of alternative nitrification processes, successful full-scale applications and relevant operational information and experiences;
- Life cycle cost of treatment process and/or alternative upgrade measure(s).

**Flow Control**

*Wastewater distribution uniformity and application dosages (trickling filter Spülkraft[1] [SK] set points)*

- Influent distribution uniformity measures with attached-growth systems, especially for trickling filters, RBCs and fluidized-bed reactors to avoid localized media overloading and excess biomass accumulation that could be detrimental to nitrification efficiency. Trickling filters may benefit from variable-speed drives, electrical rotary distributor and SK set points variation. RBCs with several trains may benefit from low headloss influent channels perpendicular to the flow path of parallel trains and influent weirs at the front of the first passes.

*Flow patterns*

- Flow patterns and direction adjustment flexibility or reconfiguration within the existing process. This factor is particularly important and applied in activated sludge process configurations such as step feed, oxidation ditch (Bio-Denitro and Triple-Ditch) and predenitrification (Chapter 5 and Appendix A).

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[1] Trickling filter dosing intensity factor.

**Preliminary Treatment**

*Fine to medium screens*

- Attached-growth systems require efficient upstream fine to medium screening to reduce clogging potential and solids accumulation within the media bed and to minimize cleaning and solids control operations such as backwashing frequency with the aerobic submerged fixed-bed reactors (BAF).

**Primary Treatment**

*Chemically assisted primary treatment*

- Chemically assisted primary treatment can facilitate the design and operational performance of a new or existing nitrifying secondary treatment process due to the reduced $cBOD_5$:TKN ratio in the effluent. However, consideration may have to be given to possible alkalinity reductions to levels below nitrification requirements.

*Control of sludge content*

- Effective sludge removal and blanket level control are required for the proper operation of primary clarifiers.

**Secondary Treatment**

*Oxygen requirement for nitrification, aeration/ventilation capacity, oxygen transfer rates and efficiency, air diffuser retrofit and aeration cycle time control*

- Oxygen demand increases due to biological nitrification activity. Total ammonia oxidation requires 4.6 kg of oxygen per kilogram of ammonium-nitrogen (Chapter 4), which may represent a significant increase, approaching 100% or more according to the influent TKN load (USEPA, 1993; WTC and VPISU, 1994). The existing type and configuration of aerators, the aeration capacities and oxygen transfer rates at nominal and peak flow conditions may not be adequate to supply the required oxygen for the nitrification process. This significant oxygen requirement may necessitate aeration system upgrading, including blowers, increased capacities with supplemental air diffusers, increase in oxygen transfer rates by retrofitting to more efficient aeration devices such as fine-bubble air diffusers, and pure oxygen supplementation. In some cases, such as SBR and dual/triple oxidation ditch, aeration cycle time control may provide a longer aerated period to favour nitrification process completion.

*Pure oxygen supplementation*

- Pure oxygen may be added to the system through *in situ* oxygenators. Pure oxygen can be supplied by pipeline or tank delivery or from an on-site air separation system for larger applications. Oxygen utilization efficiencies can vary greatly, which can have a significant impact on operating costs.

*Type of process versus types of aerators/mixers*

- The type of process upgrade may preclude the use of certain types of aerators or mixers. For instance, free-moving attached-growth support media require submerged air diffusers to promote defined mixing patterns within the aeration tank. Furthermore, only low-shear aeration and/or mixing devices can be used to prevent damage to the media.

*Dissolved oxygen concentration and aeration control: suspended growth*

- DO concentration in the suspended-growth mixed liquor is typically maintained at a minimum of 2 mg/L with aeration control (OMOE, 1984). However, nitrification may be achieved at a DO concentration lower than 2 mg/L with elevated SRTs (USEPA, 1993).

### *Dissolved oxygen concentration, aeration control and nitrification rate: attached growth*

- DO concentration in the liquid phase of attached-growth systems is typically higher than 3 mg/L to favour oxygen mass transfer across the liquid film and oxygen diffusion into the biofilm attached to the medium, thus ensuring oxygen availability to the nitrifiers. In this sense, the nitrification rate is oxygen transfer limited for total ammonia-nitrogen concentrations typically higher than 5 mg/L. The total ammonia-nitrogen concentration range at which the nitrification rate becomes ammonia limited rather than oxygen transfer limited is reported as 2–5 mg/L (USEPA, 1993).
- In attached-growth systems, increasing DO concentrations in the bulk liquid phase through aeration control can enhance the specific nitrification rate and treatment capacity.

### *Sludge retention time, media surface area and hydraulic retention time: impact on nitrification efficiency*

- SRT, media surface area for attached-growth aerobic biofilm and HRT increases are associated with biological nitrification upgrading to provide sufficient nitrifying organisms and process reaction time to meet the effluent total ammonia-nitrogen requirements.
- Increases in SRT can compensate for potentially limiting process conditions, such as low temperature, in providing more nitrifying organisms to compensate for the lower nitrification rate.
- SRT can most readily be modified in suspended-growth activated sludge processes.
- SRT is difficult to adjust in attached-growth processes that rely on the amount of active aerobic biomass inventory on the media surface and on hydrodynamic conditions within the reactor. Excess attached-growth biomass accumulation on the media surface is controlled, to the degree possible, to obtain thin aerobic nitrifying biomass.
- HRT may be increased through suspended- or attached-growth aeration tank expansion. Attached-growth aeration tank expansion may include medium- or high-density media surface area according to nitrification requirements.

### *Attached-growth support media replacement*

- Attached-growth media can be replaced with medium- or high-density specific surface area ($m^2/m^3$) to enhance the available surface area for nitrifying biomass growth. Attention should be provided to clogging with the high-density media depending on the location of the media in the liquid train (Section 5.1.4, i.e., attached-growth processes).

### *Excess biofilm attached-growth control: trickling filter SK set points, shaft rotation control, backwash frequency and mixing turbulence velocity*

- Excess attached-growth biomass accumulation control is required to favour a thin aerobic biofilm to enhance nutrient (e.g., ammonium-nitrogen) and DO diffusion. Thus, biofilm control allows efficient nitrification activity. Efficient biofilm control is obtained with attached-growth technologies by using different techniques to slough off the excess biomass according to the type of process, such as:
    - trickling filters: variable-speed drives, electrical rotary influent distributor and SK set point variation;
    - RBC: shaft rotational speed increase, reversed shaft rotation direction and RBC

train with excess growth isolation for biomass starvation to reduce biofilm thickness, supplemental aeration to assist biomass sloughing, chemical cleaning (if necessary);

- aerated submerged fixed bed (BAF): filter media bed backwashing frequency;
- aerobic submerged mobile bed, fluidized bed: airlift velocity; and
- aerobic submerged mobile bed, moving-bed biofilm (MBBR): media mixing velocity from coarse-bubble aeration system.

*Low temperature conditions: nitrification efficiency, sludge inventory, SRT and excess biomass growth control*

- Cold climate conditions will require seasonal operation modifications of the biomass inventory to maintain nitrification efficiency with suspended- and attached-growth systems according to the prevailing process temperature. SRT and MLSS concentration adjustments may be applied with suspended-growth systems. In the case of attached-growth systems, excess biomass growth control adjustments should be applied, as the solids will tend to accumulate and reduce the effective aerobic surface area. The following should also be considered:
  - Sufficient aeration tank volume with suspended- and attached-growth configurations are required to effectively accommodate the higher level of biomass inventory during cold temperature conditions to meet the effluent total ammonia-nitrogen requirement.
  - The aeration tank volume required to achieve a final effluent total ammonia-nitrogen concentration of less than 5 mg/L will be less than for a system designed to achieve complete nitrification (i.e., <0.5 mg/L).
  - Effluent quality during warmer conditions could be expected to be superior to that during cold-weather operations.

*Treated effluent recirculation with attached growth: wetting efficiency and filtration velocity*

- Treated effluent recirculation with trickling filters may be required to ensure maximum wetting efficiency of the support media to increase the aerobic biofilm effective surface area. Effluent recirculation tends to reduce patchy biofilm and dry spots on the media, especially in the lower nitrifying zones. Aerated submerged fixed bed (BAF) may require treated effluent recirculation to maintain a minimum or favour a constant filtration velocity through the media (Section 5.1.4, i.e., attached-growth processes).

*Return stream loads from solids processing and sludge management*

- Intermittent sludge management/processing and associated return stream loads may adversely affect biological reactors. High levels of total ammonia-nitrogen could be encountered in return streams, especially from anaerobic sludge digestion. Thus, equalization and controlled recycle of the return stream loads from solids processing should be considered (USEPA, 1993).

*Inhibitors and high influent organic loads*

- Separate-stage nitrification may provide an inherently greater degree of protection to the sensitive nitrifying microorganisms by removing the majority of the organic loading and inhibitory substances in the first stage, which should lead to enhanced process stability. However, complexity and cost will increase with this configuration.

*Scum formation and biological nutrient removal*

- Scum formation on the surface of suspended-growth biological reactor basins in nutrient removal processes (biological nutrient removal) may be addressed by implementing free surface discharge to evacuate the scum from the biological basins to the secondary clarifiers. The scum removal and handling system could be upgraded to avoid reintroduction of scum and filamentous organisms to the MWWTP.

**Secondary Clarification**

*Control of sludge content*

- Effective sludge removal and blanket level control are required for the proper operation of secondary clarifiers.

*Suspended growth: sludge settling characteristics, sludge bulking, rising sludge, selector zone and pre-/postdenitrification*

- Secondary clarification capacity may have to be increased to accommodate higher solids loading rates associated with suspended-growth SRT and MLSS levels required for enhanced nitrification activity, especially during low wintertime temperature conditions. Reduced sludge settling characteristics due to low F:M bulking and/or rising sludge from denitrification and nitrogen gas formation within the settled sludge may necessitate control measures. This can include the use of a selector zone to minimize the amount of filamentous organism growth and/or anoxic zones to achieve some degree of pre-/post-denitrification prior to final clarification.

*Attached-growth sludge settling characteristics, rising sludge and pre-/post-denitrification*

- The settling characteristics of attached-growth sloughed biomass are typically sufficient to maintain good settling efficiencies. Pre-/postdenitrification could be considered to reduce rising sludge potential. Denitrification configuration could be achieved with aerobic submerged fixed-bed reactors and MBBR (Section 5.3.1, Biological Denitrification).

**Operation**

*Process monitoring and control*

- A greater degree of operator attention is typically required for the monitoring and control of nitrification systems, particularly for biosolids management and SRT control in suspended-growth systems.

*Suspended growth: SRT and WAS control, sludge management and RAS control*

- For suspended-growth systems, maintenance of SRT to minimum target levels requires proper control of sludge wasting rates (WAS) and sludge settling properties. This should take into account seasonal influences on required SRT due to temperature and possible influent effects. Proper sludge management must be practised through the control of sludge recycle rates (RAS) to minimize sludge accumulation in the secondary clarification stage. Adequate instrumentation should be in place to provide accurate measurements of RAS and WAS flow rates and mass removal to facilitate sludge management.

*Attached growth: excess biofilm growth control and process stability/flexibility*

- Excess biofilm in attached-growth systems must be controlled in the nitrifying configuration to favour a thin biofilm that will maximize aerobic biofilm conditions and enhance oxygen and ammonia diffusion into the biofilm for nitrification. Attached-growth nitrification systems may provide an increased degree of reliability and stability, especially with regards to hydraulic and organic load variations, as the biomass is attached to the medium and is less susceptible to washout.

*Sludge processing recycle streams*

- Storage and controlled return of anaerobic digester supernatant and other return streams from solids processing will equalize loadings of total ammonia-nitrogen and organics to the nitrification system to enhance process stability. This is important at smaller secondary treatment plants where sludge processing operations are discontinuous in nature. Equalization of the return streams should be considered depending on the return stream loading rates and the capacity of the biological process.

*Temperature, pH and alkalinity*

- Prevailing process temperature, pH and alkalinity may have significant impacts on biomass activity and nitrification efficiency and should be monitored and controlled (Chapters 4 and 5). These considerations apply to all biological nitrification processes.
- Cold temperature will lower the nitrification rate. Thus, the treatment system efficiencies should be verified and/or piloted during the critical colder periods where effluent limitations are required (e.g., <5 mg/L target total ammonia-nitrogen concentration). Process considerations under cold-temperature operation must be evaluated and are discussed for some processes described in Chapter 5.
- The process startup for the nitrifying biomass establishment should be conducted during the higher temperature period.
- Variation of pH may be detrimental to biological nitrification efficiency as the pH is reduced from 7.0 to 6.0 (WEF, 2000b) (Chapter 4). Alkalinity should be added to systems experiencing low pH levels or pH reduction to maintain a residual alkalinity between 50 and 100 mg/L (as calcium carbonate). Denitrification can add alkalinity back into the system (Chapter 4 and Section 5.3.1.1).
- In cases where large quantities of metal salts must be added for phosphorus control, alkalinity will be consumed, and significant alkalinity supplementation may be required.

*Lagoon effluent: Tertiary nitrification flow pattern*

- A low level of total ammonia-nitrogen in lagoon effluent entering a tertiary nitrification posttreatment process during warm summertime periods may require either idling of the tertiary treatment process or provisions for the treatment of a fraction of the raw wastewater to sustain the biomass nitrification activity during this summertime period. With multicell mechanically aerated lagoons, the add-on nitrification process could be configured to treat the effluent from the initial aerated cells and discharge into the downstream facultative or aerated lagoon cells. The lagoon effluent applied to the tertiary nitrification process would contain a significant total ammonia-nitrogen load to sustain nitrification activity but would have a low effluent BOD:TKN ratio and suspended solids concentration.

*Periodic influent flow reversal to the final stages of a nitrifying RBC*

- Periodic reversal of the influent flow path to the final nitrifying stages of a multistage RBC process can promote the development of a more uniform nitrifying biofilm on each stage. Ammonia-laden sidestreams (e.g., from an anaerobic sludge digestion process) could be introduced to promote the growth of nitrifying attached-growth biomass in these lower loaded zones.

*On-line instrumentation and control benefits*

- On-line instrumentation and control may enhance process operation and flexibility and the adjustment of key process parameters (e.g., DO control, RAS flow rate and WAS flow rate control for SRT). On-line data acquisition can increase treatment plant monitoring quality and facilitate process operation. On-line control associated with an oxygen uptake rate system could identify the presence of peak organic loads or nitrification inhibitors in the influent to the treatment system. Optimum conditions for the nitrification process may be maintained, and energy savings could be achieved (WTC and XCG Consultants Ltd., 1992).

#### 6.2.2.2 Site-Specific Factors

Selection of MWWTP upgrade technologies and approaches will depend on site-specific factors related to the existing situation at existing or foreseen MWWTPs. Some of the site-specific factors are briefly summarized and discussed in the following list. The site-specific considerations often represent a critical step in the selection of appropriate upgrade measures and alternative treatment processes (USEPA, 1993). It should be noted that a varying number of these factors would apply in different cases according to the complexity of the situation.

Some of these site-specific factors are mentioned as direct upgrade measures in the selection matrices in Tables 6.1 and 6.2, while others are site-specific factors.

**General**

*Factors applying to all existing on-site treatment plants for ammonia removal and upgrade alternatives identification approach*

- Many site-specific factors are similar to the process-specific factors previously mentioned, as some site considerations have a direct impact on the type of processes that could be applied for ammonia removal;
- Regulated effluent discharge limits (e.g., total ammonia-nitrogen);
- Raw wastewater characteristics;
- Presence of an existing on-site treatment system and verification of the current level of organic and total ammonia loadings and removal capacity;
- General arrangement, design, geometry and capacity of existing treatment plant;
- Upgrading of existing on-site treatment system depends on process unit retrofit and expansion possibilities;
- Sludge disposal options and issues;
- Capital and O&M cost level that can be afforded by the municipality (including utilities costs and energy source issues).

**Influent Characteristics**

*Raw wastewater characterization*

- Raw wastewater characterization should include the following parameters:
  - average, maximum and wet weather flow rates;
  - COD, BOD$_5$, TSS, TKN, total ammonia-nitrogen, total phosphorus, alkalinity, temperature (seasonal variations), pH (note that for biological nutrient removal systems, other characterization parameters are required).

*cBOD$_5$:TKN ratio*

- Site-specific cBOD$_5$:TKN ratio has an impact on the nitrification process. A high cBOD$_5$:TKN ratio favours a heterotrophic biomass population for the organic load removal. The fast-growing heterotrophic organisms can monopolize the available oxygen. Following the reduction of the organic load, the decrease in competition for oxygen between heterotrophs and nitrifiers enhances the autotrophic nitrifier growth, and the nitrification process is initiated. Minimum required BOD$_5$ and soluble BOD$_5$

concentrations reported to achieve tertiary nitrification in attached-growth processes are discussed in Appendix A.

*Industrial wastewater*

- A potential problem for nitrifying systems is frequent and sudden changes to influent loading. Examples include discontinuous or seasonal industrial effluent discharges to the MWWTP or periodic additions of landfill leachate.
- Industrial effluent discharges to the municipal sewer may contribute a substantial fraction of the total influent load to the treatment plant. Industrial effluent discharges should be characterized to determine the organic (COD and $BOD_5$) and total ammonia-nitrogen load contribution. Inhibitory substances, such as certain organic substances and metals, can be discharged from industrial activities, and deleterious effects on the sensitive nitrifying microbial populations may occur (Chapter 4 and Section 7.2.3).
- Small MWWTPs can be strongly affected by industrial loads.

*Equalization of contaminant and hydraulic loads*

- Equalization of peak contaminant and hydraulic loads, including process solids recycle streams, should be provided if necessary to ensure lower discharge rate of the load to the plant to minimize shock impacts and overloading.

*Ammonia-laden streams*

- If anaerobic digester supernatant is available, the ammonia-laden return stream could be stored and recirculated to the nitrifying process during periods of low influent total ammonia-nitrogen to favour consistent ammonia removal.

### Effluent Discharge Requirements

*Effluent total ammonia-nitrogen requirement*

- Results of receiving water body environmental studies to establish the applicable effluent contaminant loading limits are necessary to define the need for, and degree of, required nitrification. Provincial or national standards for a mixing zone assessment should be determined.
- Effluent discharge limits applicable to the MWWTP for total ammonia-nitrogen and total nitrogen are required on monthly average, daily peak and seasonal bases. The effluent discharge limits applied should reflect the total ammonia-nitrogen discharge contribution in the nutrient budget of the receiving water body and potential toxicity impacts. The nutrient budget evaluation may influence the expenditure of funds for nutrient removal where little or no benefit can be expected (WEF, 1998). The level and type of upgrade alternative that are suited for a specific site can be evaluated based on the required discharge limits.
- Effluent discharge limit concentrations for total ammonia-nitrogen will have an impact on the total ammonia-nitrogen loading rate, aeration system capacity and biological reactor volume.

### Existing On-Site Treatment Situation

*Audit of existing treatment plant: debottlenecking, capital and O&M cost optimization*

- A conventional upgrade is based on nominal organic and/or hydraulic design capacities. A wastewater treatment plant audit (Section 6.3.2.1) with on-line monitoring equipment permits the evaluation and determination of the maximum capacity of the treatment plant and identifies the process bottlenecks that reduce the plant capacity (WTC and XCG Consultants Ltd., 1992; CG&S and Hydromantis, Inc., 1996). The MWWTP audit will help to identify cost-effective upgrade optimization, retrofit and expansion alternatives to solve process bottlenecks and enhance ammonia removal. Anticipated capital expenditure may be deferred or eliminated following the results of the MWWTP plant audit (WTC and XCG Consultants Ltd., 1992). Potential capital and O&M cost savings may be obtained through plant audit activity that far outweigh the cost of the audit.

*Space limitation and modular process additions*

- Space limitations at the existing or alternative treatment sites may lead to compact (small footprint) and modular technology evaluation, such as SBR or aerated submerged fixed-bed reactor (BAF). As an example, aerated and facultative lagoons would not represent a feasible option in cases of high wastewater flow rates and space limitations.

**Operation**

*Non-design factors limiting plant performance*

- Existing treatment plant performance limitation due to non-design-related factors, including general lack of understanding of the fundamentals of the sewage treatment processes by the plant operators, inadequate plant staffing, lack of support from administrators responsible for the facility (WTC and XCG Consultants Ltd., 1992), insufficient on-line instrumentation, inadequate control parameters and analysis frequencies, insufficient number and inadequate location of sampling points and sampling procedures.

*O&M level of difficulty versus size of the municipality*

- Level of operator attention, process complexity and/or high operation and monitoring requirement may preclude some process technology alternatives, such as fluidized-bed reactors, for small municipalities and remote communities. Low operating and monitoring demand technologies, such as aerated lagoons with downstream tertiary nitrification processes, including trickling filters, recirculating and intermittent sand filters, constructed wetlands and others (Chapter 5), may be more applicable.

## 6.3 Final Selection of Facility Upgrade Measures

The most appropriate upgrade measures to be implemented in each application will depend on the factors that are limiting the capability of a particular MWWTP to control total ammonia-nitrogen consistently. Through an initial assessment and prioritization of site-specific and process-specific factors, as outlined in the previous section, various upgrade approaches can be identified and narrowed down. This usually requires the assistance of engineering consultants throughout the various phases of the project, which will lead to the selection of the most appropriate technical solution and ultimately to its installation and/or implementation.

External engineering consulting firms can assist the municipal engineer in defining specific upgrade requirements and solutions, starting with a long list and leading to a subsequent short list of alternatives for evaluation, taking into account the various project-specific factors of importance. Proper conceptual designing of alternative processes is essential. In many cases, a class environmental assessment or similar procedure will be required, which will necessitate an evaluation of numerous possible remedial options.

### *6.3.1 Phases of a Major Wastewater Treatment Project*

The typical steps that a municipal engineer and community management will have to address and coordinate in the development of a major wastewater treatment plant installation or facility upgrade project, together with the selected external consultant(s), can include:

1. Opportunity identification (i.e., definition of effluent discharge requirements/objectives and time frame based on more stringent discharge regulations);
2. Project conception/initiation, including setup of project management and a planning committee;
3. Project planning and public consultation;
4. Development of terms of reference for engineering consultant work in the various project stages, followed by the preparation of a request for letters of interest and/or request for proposals and subsequent evaluation and selection of a consultant(s);
5. Receiving water studies (e.g., including assimilative capacity assessments);
6. Determination of effluent discharge requirements (i.e., effluent quantity and quality criteria and corresponding treatment requirements);
7. Desktop analysis for assessment of existing treatment plant processes;
8. Audit and evaluation of the existing treatment plant processes;
9. Class environmental assessments, including the public consultation process;
10. Evaluation of short list of upgrade options for the existing plant or new treatment process options, including associated life cycle cost assessments;
11. Pilot plant studies;
12. Preliminary conceptual designs and order-of-magnitude costing of appropriate upgrade options;
13. Selection of plant upgrade measure(s) or new treatment processes;
14. Facility planning and reporting;
15. Preliminary engineering design and report;
16. Project approvals process;
17. Detailed final design and specifications;
18. Financing plans;
19. Construction; and
20. Startup, commissioning and operation of new treatment process installations.

The planning and process selection phase of a wastewater treatment project is considered to be complete following the project approvals stage (i.e., Step 16 above — namely, the acceptance of the facility plan and/or preliminary engineering report). Subsequently, the implementation of recommendations from the project planning phase will begin with the preparation of a formal engineering design (i.e., Step 17 above). Additional details on the specific requirements and activities involved in the various phases of a major project are described in USEPA (1992) and Great Lakes – Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers (1997).

### 6.3.2 Techniques to Assist in the Selection of Plant Upgrade Options and Processes

Prior to the upgrading of existing MWWTPs for enhanced treatment performance, an assessment and evaluation of the existing processes should be conducted. Various techniques and approaches exist to accomplish this, some of which are described below.

As described in Section 6.1.2, the capacity and treatment capabilities of existing plants can potentially be increased through a variety of techniques, including physical expansion or, alternatively, and where applicable, less capital-intensive approaches, such as the upgrading of

O&M procedures and process control strategies and/or minor physical modifications.

The typical plant upgrading approach involves the following sequential steps (Daigger and Buttz, 1998):

1. Desktop analysis;
2. Identification of performance-limiting factors;
3. Identification of upgrade options;
4. Evaluation of upgrade options;
5. Selection of upgrade option; and
6. Implementation.

### 6.3.2.1 Process Audits

To address the potential for capacity and performance maximization at existing MWWTPs, Environment Canada and the Ontario Ministry of the Environment have, since the mid-1980s, promoted a methodical approach for plant optimization, debottlenecking and capacity assessment — namely, the process audit (Nolasco and Manoharan, 1999; WERF, 1999). This consists of a variety of desktop and field testing analysis tools for the determination of ultimate plant capacity and/or enhanced treatment performance capabilities. These techniques are described in a comprehensive guidance manual ("Guidance Manual for Sewage Treatment Plant Process Audits" by CG&S and Hydromantis, Inc., 1996) that is available from the Ontario Ministry of the Environment.

The wastewater treatment plant process audit is a full-scale plant performance assessment methodology, covering Steps 1 and 2 in the plant upgrading approach outlined above. Process audits have been proven to be beneficial in helping to determine the ultimate capacity of existing MWWTPs and the potential for treatment performance enhancements (e.g., to examine the ability of existing MWWTPs to meet possible future more stringent requirements for consistent year-round nitrification). The plant audit approach can help to troubleshoot design and process control problems and to identify system bottlenecks in the liquid and solids trains that can reduce plant capacity.

Based on audit findings, low-cost modifications may be identified to enhance plant performance (i.e., Step 3 above). This can include the optimization of plant operations, upgrading of existing equipment for enhanced performance and implementation of energy conservation measures. Some of the benefits that can result from a process audit include avoidance of unnecessary and costly plant expansions, the ability to meet more stringent effluent discharge quality requirements and reductions in operating costs (CG&S and Hydromantis, Inc., 1996). Plant audit findings can be used to indicate the requirement for major facility modifications and to identify appropriate upgrade options, such as plant expansions or the construction of new replacement processes (WTC and Process Applications Inc., 1996).

#### 6.3.2.1.1 Process Audit Technologies

The following is a list of typical process audit technologies and techniques (CG&S and Hydromantis, Inc., 1996):

- historical data review;
- on-line real-time monitoring of important process parameters;
- off-line monitoring for the characterization of process streams;
- field measurements such as aeration efficiency and oxygen transfer rate analysis;
- stress testing for the evaluation of unit process performance during peak loading conditions;
- tracer testing to determine flow patterns and flow short-circuiting;

- hydraulics analysis through measurements and modelling;
- dynamic process and plant modelling and simulation; and
- general process analysis through the review of overall plant processes (e.g., analysis of operational data, monitoring and control strategies, etc.).

Figure 6.2 depicts a sample task flow sheet for a plant optimization project making use of various process audit technologies (CG&S and Hydromantis, Inc., 1996). Projects will vary in scope depending on the case-specific requirements and budget. This has to be considered in the selection of tasks and audit technologies. Project tasks are typically phased to permit the examination of plants on a step-by-step basis.

Brief descriptions of some of the major audit technologies (i.e., on-line monitoring, hydraulic analyses, plant stress testing and oxygen transfer efficiency determinations) are provided below (CG&S and Hydromantis, Inc., 1996; Daigger and Buttz, 1998; Nolasco and Manoharan, 1999). These can all potentially relate to the assessment of existing MWWTPs for nitrification capabilities.

#### 6.3.2.1.2 On-line monitoring

On-line monitoring is a system consisting of field instrumentation interfaced to a computer to collect and record "real-time" data. On-line monitoring is a key component of a process audit and provides information on the dynamic behaviour of the treatment processes under typical operating conditions or in association with other process audit tasks (CG&S and Hydromantis, Inc., 1996).

The following are some of the objectives of on-line monitoring in the context of a process audit:

- to reveal real-time responses by providing an instantaneous display of data;
- to determine cause-and-effect relationships within the treatment plant;
- to provide a system for data storage and statistical data analysis; and
- to support data collection requirements for other process audit technologies or process modelling.

#### 6.3.2.1.3 Hydraulic analyses

The hydraulic capacity and flow distribution characteristics of a plant are major factors affecting capacity and operational performance and control. The measurement and simulation of plant hydraulics are a valuable tool to identify key bottlenecks in the system. This can be used to optimize and improve treatment plant performance, often at relatively low cost.

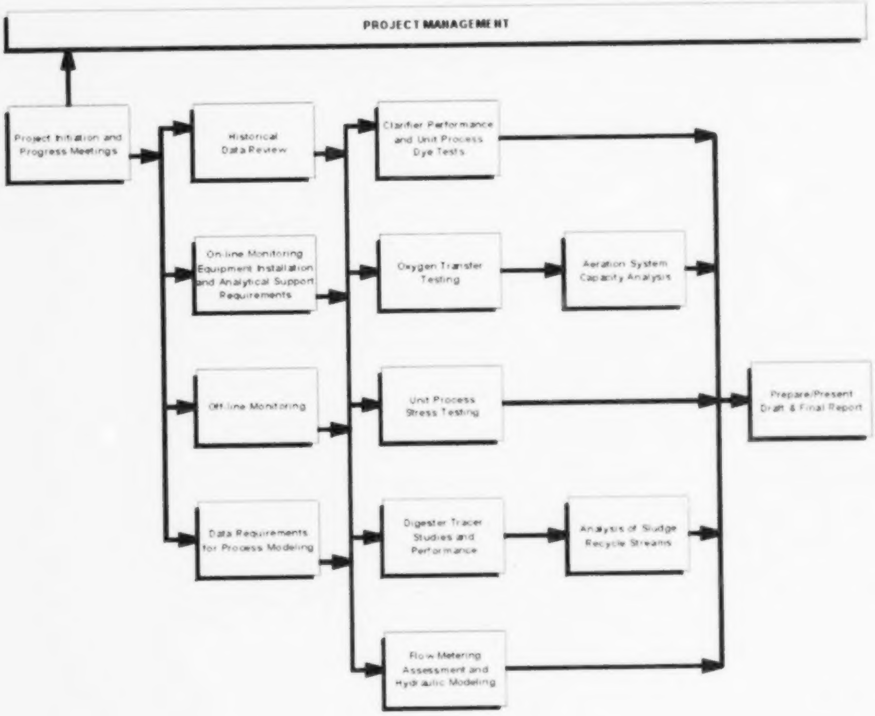
Hydraulic testing and analysis require a step-by-step approach, including:

- verification of as-constructed drawings;
- field survey to establish levels and benchmark;
- definition of hydraulic characteristics, including the elements and equations used to describe them; and
- hydraulic model calibration at known flows, often with the use of on-line data collection, and modelling or simulation of required flows.

*Hydraulic modelling can be used to:*

- determine the hydraulic capacity of an existing facility;
- identify hydraulic bottlenecks and capacity-limiting factors and to investigate alternative strategies for reducing these limitations;
- determine flow imbalances and investigate methods to improve the flow distribution between parallel unit processes; and

**Figure 6.2 Sample Layout of Tasks for the Management of an MWWTP Process Audit and Optimization Program**


PROJECT MANAGEMENT
Project Initiation and Progress Meetings
Historical Data Review
On-line Monitoring Equipment Installation and Analytical Support Requirements
Off-line Monitoring
Data Requirements for Process Modeling
Clarifier Performance and Unit Process Dye Tests
Oxygen Transfer Testing
Unit Process Stress Testing
Digester Tracer Studies and Performance
Flow Metering Assessment and Hydraulic Modeling
Aeration System Capacity Analysis
Analysis of Sludge Recycle Streams
Prepare/Present Draft & Final Report

- determine velocity and energy gradients and identify optimum locations for chemical addition.

#### 6.3.2.1.4 Stress testing

Process unit stress testing can be used to demonstrate the full-scale impact of increased hydraulic, organic and solids loading rates and to demonstrate the potential for debottlenecking and rerating. Stress testing, normally conducted with on-line monitoring, is used to increase the loading rates to a unit process beyond normal operations. The response of the unit process to variation in the loading rate is quantified and the results are presented as process loading condition versus corresponding performance characteristics. Stress testing provides a quantitative measure of the impact of elevated loading rates on process performance to determine actual capacities of unit processes.

Stress tests are feasible where parallel units are in service and one or more units can be temporarily removed from service or their loading reduced. Stress testing may result in the deterioration of effluent quality; therefore, the project and plant staff should be well informed and the regulatory agency advised of the work as required.

During stress testing, on-line and off-line monitoring of process responses should be intensified to provide a complete description and characterization of performance. However, with stress testing, it is not usually possible to test all conditions that may occur in a facility — for example, changes in sludge settleability. Therefore, attention to risk must be considered to ensure that results can be extrapolated to other conditions that can occur. In addition, modelling of conditions that may occur can be used to evaluate many operating scenarios that may not be experienced in short-term or even long-term stress tests.

### 6.3.2.1.5 Oxygen transfer testing

Oxygen transfer rate is a key factor in estimating maximum aerobic treatment plant capacity for the removal of organic matter and for nitrification. Oxygen transfer rate and efficiency are site specific and can vary significantly from plant to plant (CG&S and Hydromantis, Inc., 1996). The oxygen transfer testing technology therefore forms a critical component of a plant performance audit, particularly as the requirement for enhanced ammonia reduction at MWWTPs becomes mandated. More precise information on plant oxygenation capacity can result in improved energy efficiencies through possible aeration system retrofits and/or DO control.

Protocols for "dirty water" and "clean water" testing have been established for the measurement of oxygen transfer efficiency for aeration systems (USEPA, 1989a; CG&S and Hydromantis, Inc., 1996; Daigger and Buttz, 1998). Techniques include the off-gas and hydrogen peroxide methods of determining oxygen transfer rate as a function of power input or diffused air flow rate. These two methods have been widely used to evaluate the potential for energy savings through on-line DO control and subsequently for capacity analyses.

The investigation of aeration capacity can identify opportunities for plant capacity increases without major capital construction (e.g., expansion of aeration tankage). This can include an upgrading of the existing aeration system (e.g., aerator retrofits) and/or reduction of the aeration influent loading — for example, by enhanced primary treatment for enhancing nitrification.

Regardless of the type of aeration system, the variability in actual oxygen transfer from facility to facility can be significant. Some surface aeration systems have reasonable transfer efficiencies and can be used in nitrification processes. A case-by-case assessment and cost–benefit evaluation should be performed prior to deciding on a conversion to fine-pore aeration, which has been a popular recent approach.

### 6.3.2.1.6 Example of an MWWTP Process Audit and Evaluation

A previously conducted comprehensive evaluation of the City of Windsor's Little River MWWTP serves as a good example of the use of the process audit approach. A number of techniques were employed for determining actual plant capacity and the capability of achieving year-round nitrification to meet more stringent wastewater discharge limits. A description of the Little River MWWTP, the process audit techniques and optimization measures utilized, and the important results obtained are reported as a detailed case study in Chapter 9.

### 6.3.2.1.7 Information Sources

The guidance manual for conducting process audits at MWWTPs (CG&S and Hydromantis, Inc., 1996) provides detailed descriptions of the technologies and methodologies available for conducting process audits. It includes guidance for project setup, general resource requirements and overall budgeting and contains case study examples of full-scale applications and sample calculations and data sheets. This useful reference source is intended to assist individuals responsible for the assessment, optimization and upgrading of treatment plants. Particular emphasis is given to facilities that employ conventional or modified activated sludge processes. An understanding of wastewater treatment theory and applications is required for use of the manual.

Process audits can help to determine actual plant capacity as it relates to the physical facilities and can test various process control strategies. However, O&M procedures and plant management issues are not typically analyzed. Therefore, process audit results should be considered in conjunction with other approaches for the assessment of the latter plant capacity factors. Accordingly, the guidance manual for treatment plant process audits (CG&S and Hydromantis, Inc., 1996) is considered to be a companion document to the handbook "The Ontario Composite Correction Program Manual for Optimization of Sewage Treatment Plants" (WTC and Process Applications Inc., 1996). This handbook is derived from the U.S. EPA's published handbook entitled "Retrofitting POTWs" (USEPA, 1989a). Additional information on the Composite Correction Program (CCP) approach and process optimization is provided in Chapter 7.

### 6.3.2.2 Preliminary Experimental Testing and Pilot Plant Studies

Chemical characterization and treatability studies are often necessary to screen candidate treatment processes and operational strategies for particular applications. Furthermore, pilot studies are advisable to verify performance capabilities of novel or proprietary systems (WEF and ASCE, 1992).

There are marked variations in the maximum specific growth rate of nitrifiers between systems treating different wastewaters. This can have a substantial effect on system design (WEF, 1998). These variations often correlate with the industrial component in the raw municipal wastewater, with increased industrial input potentially resulting in increased degrees of inhibition towards relatively sensitive nitrifying microorganisms.

#### 6.3.2.2.1 Wastewater Characterization

Initial assessments of candidate wastewaters for the application of a biological nitrification process, particularly where the contribution of potentially toxic industrial effluent(s) is relatively high, should consider the following characterization parameters, including average values and typical ranges:

- flow rate;
- total and soluble COD;
- cBOD;
- TSS and VSS;
- TKN;
- total ammonia-nitrogen;
- total phosphorus;
- pH;
- alkalinity; and
- temperature.

The characteristics of treatment plant recycle streams contributing to the biological process influent should also be measured.

#### 6.3.2.2.2 Treatability and Biological Activity Testing

Biotreatability testing with representative samples of wastewater can provide important information on inhibitory effects, if any, on nitrification activity. The experimental results can be used to generate kinetic information for consideration in:

- the design of new biological treatment facilities;
- the design of new add-on processes for existing facilities; or
- the optimization of existing treatment plant process conditions.

Test methods and results are used mainly for suspended-growth systems; however, arrangements for attached-growth systems are possible.

Biotreatability and nitrification rate testing can involve:

- batch nitrification systems;
- fed-batch systems using fill-and-draw methods; or
- continuous-flow reactor arrangements.

Relatively simple bench-scale batch aerobic treatability methods can be used to test samples of wastewater under controlled conditions for the determination of nitrification kinetics. Based on the rate of oxidized nitrogen production (i.e., nitrate and nitrite) and/or ammonium depletion, the nitrification rate is calculated. Additional details on testing procedures are provided in Randall *et al.* (1992).

The impact of microbial acclimation to inhibitory substances and conditions on process kinetics should be considered (WEF, 1990; WERF, 1999). If the biomass used in batch testing is not acclimated to the actual wastewater, the results may underestimate the treatability characteristics of the wastewater. It is preferable to use acclimated sludge from a full-scale nitrifying process or from a pilot plant process treating the actual wastewater to seed the batch treatment reactor.

Alternatively, fed-batch or continuous-flow reactor arrangements can permit the acclimation of the biological sludge to inhibitory chemical substances and stress conditions. This mode of testing is more representative and can provide steady-state nitrification rate information. However, the duration of experiments and the associated equipment and experimental cost will be considerably greater.

Field pilot plant testing can effectively simulate actual site conditions, including the treatment of fresh influent wastewater supply having a more realistic degree of variability. Obtaining nitrification kinetics during pilot plant testing is desirable to determine if the candidate wastewater being treated may be inhibiting nitrification rates and if the expected nitrification process design safety factor will be met (Randall *et al.*, 1992).

Large-volume respirometers have proven useful in determining the biotreatability of candidate influent wastewaters, including the impact of certain contributing industrial effluents on nitrifying biological treatment processes (Schwing, 1999). Industrial effluent pretreatment strategies can be developed with the assistance of such wastewater treatability testing.

#### 6.3.2.2.3 Pilot Plant Testing

Pilot plant studies are particularly beneficial for the evaluation and demonstration of new and innovative treatment processes and unique process configurations, and in cases where considerable uncertainty exists with regards to nitrification activity and appropriate design SRT levels (e.g., applications involving very cold temperature conditions or where the industrial effluent contribution to the municipal wastewater is high and/or toxic). Pilot plant information generated for a particular application can be used to determine appropriate full-scale process designs including safety factors, which take into account the effects of inhibitory chemical constituents in the wastewater and/or anticipated extreme conditions of operation.

Pilot plant studies involving the evaluation of one or more processes are useful for determining and demonstrating:

- wastewater treatment performance capabilities;
- hydraulic performance;
- operational stability;
- process flexibility;
- operational requirements;
- design parameters; and
- process economics.

Randall *et al.* (1992) outline reasons for carrying out pilot plant studies specifically for biological nutrient removal process applications, which can involve relatively complex process configurations and control strategies. Some of the related study objectives include:

- determination of performance characteristics and feasibility of a particular biological nutrient removal process design;
- comparison of alternative biological nutrient removal processes to determine the most cost-effective and reliable process for full-scale design;
- to observe long-term process performance as a function of influent wastewater characteristics and variability;
- to generate information needed to establish optimal and cost-effective designs; and
- to identify safety factors for specific biological nutrient removal designs.

Field pilot plant trials provide treatment plant staff with valuable operating experiences with different processes.

In the planning of pilot plant studies, consideration should be given to side-by-side evaluations of different process types with a common influent supply to allow for useful comparative assessments. Some other important factors for consideration in pilot plant program planning include:

- process designs and configurations;
- level of on-line process monitoring and control;
- process loading and operating conditions to be tested; and
- sensitivity analysis for critical process parameters.

Pilot plant designs should reflect closely the actual design of the candidate full-scale process and should be large enough to provide meaningful data to facilitate the scaling up of results. Pilot plant testing should generate sufficient volumes of treated effluent sample for biological characterizations, such as bioassay testing for aquatic toxicity effects. Samples of acclimated pilot plant process sludge can be used as biomass inoculum for batch nitrification tests.

Experimental plans for pilot plant studies usually involve comprehensive sampling and chemical analysis programs, as replicate sampling and analysis are often required to ensure the reproducibility and reliability of the data generated. Instrumentation for on-line process monitoring and chemical analysis can be beneficial.

An important consideration in determining the duration of pilot plant trials is that long periods are often required to establish steady state for each defined experimental period (i.e., set of applied process conditions). A minimum period of at least 2–3 times the process SRT is usually required, which can be lengthy for nitrifying systems. Furthermore, pilot plant investigations are sometimes conducted over a period of at least 1 year to permit the assessment of seasonal effects on treatment process performance.

In certain cases, it is useful to conduct field pilot plant trials in parallel with an existing full-scale secondary treatment plant to assess potential upgrade options such as process optimization measures. Advantages of conducting such pilot-scale trials instead of full-scale process modifications and evaluations include the following:

- relative ease of changing process conditions;
- lower cost;
- less risk associated with stress testing;
- ability to comparatively assess different process types and conditions of operation;
- ease of control of process conditions; and
- more accurate measurements of process kinetics and determinations of growth rate as a result of increased degree of control.

The cost of conducting a pilot plant study can be considerable and will be very case specific, depending on the type, number and size of processes to be evaluated, testing location and the duration of the planned trials. This includes the cost of fabricating or renting the pilot plants and transporting them to the test site. Furthermore, arrangements must be made at the site for the connection of influent supply and utilities and labour for project planning, supervision, day-to-day O&M, monitoring, sampling and chemical analysis.

#### 6.3.2.2.4 Pilot plant program activities

An itemized summary of important pilot plant program activities is listed below:

- problem identification;
- definition of pilot plant testing and demonstration goals;
- selection of engineering consultant to coordinate the pilot plant trials;
- identification and selection of alternative treatment processes to be tested;
- design and fabrication of pilot plant(s) and/or rental of units from process vendors;
- arrangements for equipment, instrumentation, materials and supplies;
- hook-up of utilities and influent supply;
- development of clear experimental plans, including sampling schedules and monitoring parameters;
- development of O&M strategies, plans and schedules;
- startup, operation and monitoring of pilot plant(s);
- sampling and chemical analysis;
- collection, compilation and analysis of experimental data;
- performance assessments;
- reporting of results and recommendations; and
- overall project management.

The terms of reference for selecting an engineering consultant to manage a pilot plant treatment project would be similar to standard terms of reference for hiring a consulting firm for a full-scale wastewater treatment project. The engineer assigned to manage the project would need to have relevant experience and a strong process engineering background, including a comprehensive understanding of treatment process fundamentals. A research and development background would be beneficial and a possible criterion for selection.

#### 6.3.2.2.5 Examples of pilot plant studies

A major pilot plant investigation that was conducted at the West Windsor MWWTP (City of Windsor, 1992, 1996) represented an example of a comprehensive testing program to assess various nitrification processes for potential full-scale application at the plant. One of the treatment performance targets was to achieve an effluent total ammonium-nitrogen concentration of less than 3 mg/L.

A number of innovative processes were evaluated and demonstrated at the large pilot scale at the site of the existing 159 MLD (35 MIGD) chemically assisted primary treatment plant. The nitrification systems studied included BAF, trickling filter/solids contact, RBC, modified activated sludge as well as a conventional activated sludge process that served as a control. Three different BAF process technologies were assessed. Various loading conditions and influent temperatures were evaluated. The trials lasted for over 1 year. Based on the results of the pilot plant studies, corresponding full-scale process designs and cost estimates were developed (City of Windsor, 1996).

Conducting a pilot plant study can be costly. However, the benefits can outweigh the costs. The cost of the long-term pilot plant trials conducted at the West Windsor MWWTP amounted to about $500,000. This price was considered by the client to be well justified, since the resultant recommendations for innovative plant upgrading had considerable associated cost savings compared with other more conventional upgrade options.

Another example of pilot plant testing involved the development of a biological nutrient removal process design for the Lethbridge, Alberta, MWWTP, as described by Nolasco *et al.* (1998). Owing to the complexity of the particular biological nutrient removal process employed, modelling and simulation techniques in conjunction with pilot plant testing were employed. The pilot plant was operated for about 1 year and utilized an automatic on-line data acquisition system for the monitoring of numerous parameters, such as DO, MLSS, total ammonia-nitrogen, nitrate-nitrogen, orthophosphate and flow rates. Mathematical modelling was used as an experimental design and process optimization tool. Careful experimental planning is required in such cases to ensure that data to be generated by pilot plant testing are useful and representative for model calibration (USEPA, 1993).

### 6.3.3 Selection of Engineering Consultants

The services of specialized engineering consultants may be required during the various phases and stages of a wastewater treatment project, as listed in Section 6.3.1. This will depend on the size and scope of the project for either upgrading an existing MWWTP or the installation of a new facility.

The involvement of engineering consultants in a major wastewater treatment project usually begins in the project planning phase (i.e., Steps 3–15 listed in Section 6.3.1). The role of a consultant in project planning is to facilitate the work of the project management and planning committee. The latter group is set up at the time of project initiation and is ultimately responsible for making critical decisions concerning the project. The various roles and responsibilities of management and the planning committee are described further in USEPA (1992).

Not all wastewater projects require consultants during the planning stages. Factors that dictate the need for involving a consultant at this stage include (USEPA, 1992):

- project size, with the larger projects more likely requiring the assistance of external consultants;
- project complexity;

- availability of volunteer experts as planning committee members;
- technical capability of existing project community management and planning committee members; and
- project budget.

Use of an external consultant during the planning phase eliminates potential liability for the performance of the recommended plan and the subsequent associated system design.

Following the project planning phase, paid consultants and contractors are generally used for the formal engineering design phase of a project because of the expertise required to prepare technical plans, specifications and legal documents required to secure regulatory approval and for use as construction documents. The consultant solicitation and selection process for this phase is similar to the process followed in the planning phase. These are described in greater detail in the subsection below.

#### 6.3.3.1 Request for Proposals

As indicated under Step 4 in Section 6.3.1, the solicitation and selection of consultants for the various phases and stages of a treatment project by the community project management will typically require the preparation of:

- request for letters of interest and statements of qualification;
- terms of reference for the consulting work; and
- request for proposals.

A request for proposal generally begins with a brief background on the site-specific problem and the objectives and requirements of the work to be performed by the selected consulting firm. It also defines the terms of reference for the project participants, including the consultant, and required interactions between the participants. The project schedule and budget (optional) and the specified qualifications of the consultants should be included. In some cases, guidelines for the preparation and submission of the proposal, including the format and content, and criteria for proposal evaluation and selection are provided. The remainder of the request for proposal should contain a description of other required submittal contents, such as information on insurance, liability and contractual requirements. The form of the request for proposal, including the insurance, liability and other legal issues, should be reviewed by the community lawyer before the request for proposal is issued (USEPA, 1992).

The request for proposal should request that the following items be covered in the proposals from the proponents:

- description of the problem;
- proposer's understanding of the project needs;
- proposer's approach to solving the problem;
- statement of work and a detailed scope of services to be provided;
- schedule of work;
- consulting firm's documented experiences with similar projects, including a list of references;
- proposed staffing for the project, with résumés;
- staff organization for the project;
- categories of technical and administrative staff to be assigned to the project and estimates of the respective time allocations and fees for each person; and
- proposed fee or fee schedule (optional).

A final report with findings and recommendations will likely represent the main deliverable for individual projects and work under the planning phase.

The request for proposal and responses should be much more focused for work related to the detailed design phase of the project (USEPA, 1992). Furthermore, other non-technical requirements, such as indemnities, liability insurance, bonds and guarantees, should be established with advice of the community's counsel.

### 6.3.3.2 Proposals Review, Interviews and Selection

The proposals received need to be carefully reviewed and evaluated to create a short list of firms to be considered and interviewed. Individual interviews with candidate consultants by the selection subcommittee are optional. These are carefully scheduled affairs to help differentiate the best proposal from the others. The interviews usually begin with a short formal presentation by the candidate consultants. Following the presentation, the selection subcommittee poses questions of the consultant. Based on the information received in the proposals and during the interviews, a consultant will be selected.

The establishment of a price for consultant services can be approached in several ways. This can involve a request for a price as part of the proposal submission or as part of a separate simultaneous submittal (e.g., a "price proposal"). This will include the total price and a breakdown with details, including assigned person-hours. Alternatively, the price can be negotiated following the selection of the consultant based on technical criteria.

It is noteworthy that the process of retaining a consultant may be difficult for small communities owing to the likely lack of experience in contracting for professional services. In particular, it is important to properly define the work to be done by the consultant, so that all proposals can be evaluated on a common basis. It may be difficult for the selection subcommittee to identify the firms with the most relevant experience in those technologies pertinent to the community's needs. Where government grants or loans are involved, regulations and guidelines have to be followed closely. Additional details on the various aspects of soliciting, evaluating and selecting a project consultant are described in USEPA (1992).

# Chapter 7
# Optimization, Troubleshooting and Operator Training

# Chapter 7
# Optimization, Troubleshooting and Operator Training

This chapter provides an overview of process optimization as a potential low-cost means of MWWTP upgrading to achieve consistent year-round nitrification. It includes a discussion on the importance of proper process O&M for attaining enhanced treatment performance and an overview of process troubleshooting applicable to nitrification processes.

In general, there are three opportunities to achieve capacity gains within existing treatment facilities without the requirement for expansion or the construction of additional process units — namely, process debottlenecking, rerating and optimization (CG&S and Hydromantis, Inc., 1996; WERF, 1999). The costs associated with these opportunities are by definition small compared with those of the more conventional and conservative approaches of capital expansion or new facility construction.

The potential opportunities for debottlenecking, rerating and/or optimization exist due to traditional overly conservative steady-state-based plant designs or, conversely, to design and/or operational deficiencies (CG&S and Hydromantis, Inc., 1996; WERF, 1999). The latter deficiencies can leave considerable room for treatment performance improvements at existing facilities.

In the evaluation of site-specific plant optimization opportunities, it should be taken into consideration that in certain cases, optimization measures alone may be insufficient, and that plant expansion may be necessary. Also, increased manpower and skills training are required to operate plants at higher than the original rated capacity (Nolasco and Manoharan, 1999).

## 7.1 Optimization

### 7.1.1 Background

Considerable scope exists to optimize the performance of existing MWWTPs. Optimization can involve the development of improved operational strategies, adjustments to modes and conditions of operation, the use of on-line monitoring and control and/or low-cost minor modifications to unit processes and equipment to achieve increased capacity and/or enhanced treatment performance (Daigger *et al.*, 1992; WTC and XCG Consultants Ltd., 1992; CG&S and Hydromantis, Inc., 1996; Daigger and Buttz, 1998). Considerable cost savings have been demonstrated by process optimization and/or the application of innovative retrofits (e.g., WTC and XCG Consultants Ltd., 1992; WTC and VPISU, 1994; Nolasco and Manoharan, 1999).

For nitrification applications, optimization techniques focus mainly on maximizing nitrification rates and enhancing process stability and reliability (WERF, 1999). Treatment process modelling and simulation can play an important role in determining plant optimization alternatives, identifying appropriate operating strategies and predicting treatment performance. This applies particularly for the complex biological nutrient removal process configurations. Additional discussions regarding modelling and simulation tools are provided in Chapter 8.

### 7.1.2 Approach

At the start of a full-scale plant process evaluation and optimization project, it is important to clearly define the objectives, scope and approach of the project. In general, this should involve all parties, including plant owner/operator, consultant and regulatory agency as appropriate. A general task-by-task project flow sheet should be prepared to document the approach (Nolasco and Manoharan, 1999; WERF, 1999).

The task of optimization requires an in-depth knowledge of the particular plant processes and associated technologies. In MWWTP evaluation and optimization projects, interactions between the various process stages in both the liquid and solids trains of the treatment facility are important and should be given consideration.

A plant process flow sheet should be prepared to ensure that all important plant streams, including recycle and return streams, are included. This can assist in developing the data required for a plant mass balance and avoid missing key streams. In particular, recycle streams such as the RAS, WAS, digester supernatant and dewatering system filtrates must be recognized to close overall plant mass balances as accurately as possible.

A plant mass balance is key to confirming and understanding the flow and accumulation of materials in a plant. Frequently, it is difficult to successfully close a mass balance due to inaccurate flow or sampling and analytical measurements. This can, however, assist in identifying deficiencies in flow metering and process monitoring, sampling and analysis that should be corrected. Inaccuracies will propagate through an analysis and would need to be considered in the final engineering recommendations.

### 7.1.3 Steps Involved in Plant Upgrading Through Optimization

As described in Section 6.3.2 and outlined in Figure 7.1, various steps are involved in identifying, evaluating and implementing plant upgrade options. Plant upgrade alternatives that involve process optimization and/or minor physical modifications, rather than more capital-intensive expansions and new process additions, are the focus of this section.

An initial plant assessment is an important first step in determining optimization opportunities. Initial information review and desktop analysis (i.e., Step 1) are critical to determining the focus of Step 2 (i.e., process auditing as discussed in Section 6.3.2.1). This ensures that the most efficient and effective field testing technologies are employed for plant evaluation and the identification of performance-limiting factors.

The initial plant assessment is recommended for several reasons (Daigger and Buttz, 1998), including the following:

- Historical plant performance can help establish target loading conditions for the field testing.
- Past high-loading conditions may already have accomplished a certain amount of the plant stress testing.
- Problem processes can be identified, thereby allowing the evaluator to target the field testing program.

Information should be gathered from persons most familiar with the particular MWWTP, including plant administrators and operators, design engineers, process experts and regulators. The scope of the desktop analysis depends on the size of the plant and the processes to be tested.

Results of the initial assessment and the field evaluations of existing plant design and operational characteristics obtained during Steps 1 and 2 (Figure 7.1), respectively, can be used to identify suitable optimization and debottlenecking alternatives. The field evaluations can involve the use of process audit techniques and the CCP approach described in Section 7.1.1. The identification of upgrade options can also utilize techniques such as engineering evaluations and workshops. It is important that a broad range of options be identified and screened at this stage to determine the most promising ones for subsequent detailed evaluation (Daigger and Buttz, 1998). Relative cost analyses can be used as a screening tool at this stage.

Detailed evaluations of the most promising upgrade alternatives can involve such techniques as pilot plant testing of new technologies (Section 6.3.2.2), full-scale field testing of alternative operational strategies, model predictions, surveys of facilities using similar technologies and equipment, and engineering calculations. The results of these evaluations form the basis for selection of the most appropriate upgrade option(s).

The selection process involves a comparative assessment of a short list of options, taking into consideration detailed life cycle cost analyses and technical factors such as process reliability, ease of operation, demonstrated full-scale performance, staff capabilities at the existing MWWTP and implementability (Daigger and Buttz, 1998).

The final step is the implementation of the selected upgrade measures. This can involve the installation of new equipment and/or the development and implementation of new O&M procedures and strategies. This will likely require the training of staff in the effective use of the existing facilities. A field evaluation of the upgraded plant would be worthwhile to establish baseline performance characteristics.

In summary, this general multistep plant upgrading approach ensures that consideration is given to factors that affect the capacity of the treatment plant and that effective use is made of the facility (Daigger and Buttz, 1998).

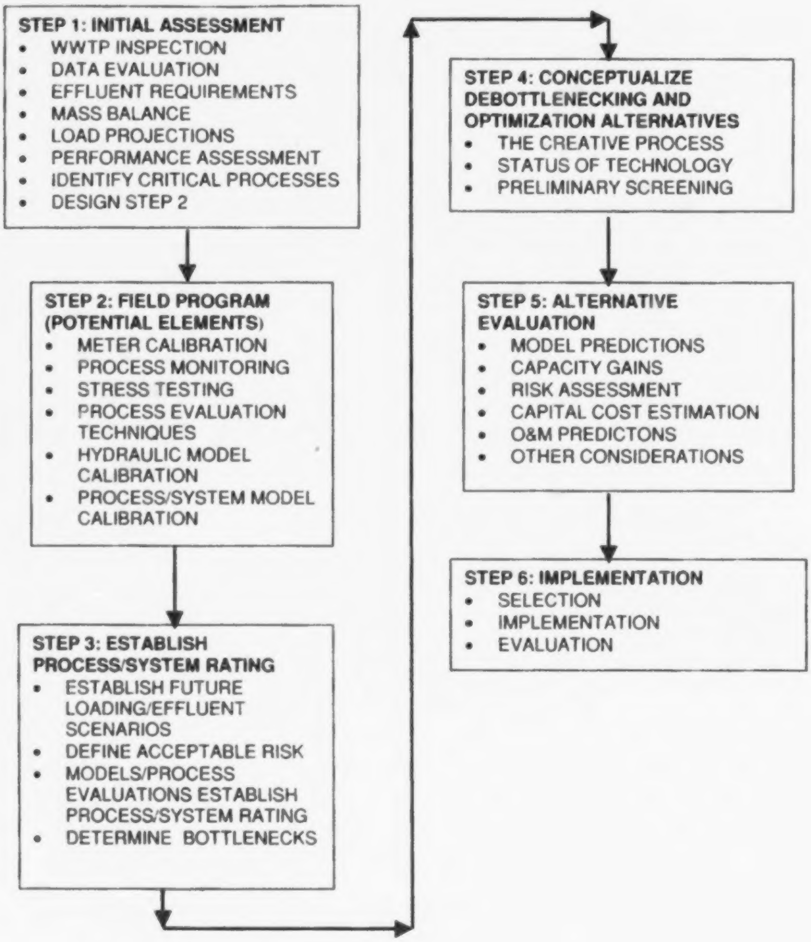
### *7.1.4 Composite Correction Program*

O&M procedures and resources and plant management practices can be important factors for consideration in plant performance evaluations and the determination of upgrade measures. Methods to identify and evaluate these potential performance-limiting factors are described in the handbook "The Ontario Composite Correction Program Manual for Optimization of Sewage Treatment Plants" (WTC and Process Applications Inc., 1996), which is derived from USEPA (1989a).

The CCP approach has been successfully applied and demonstrated in the province of Ontario for determining the potential for low-cost optimization of many existing MWWTPs (WTC and Process Applications Inc., 1996). The CCP was developed by the U.S. EPA and is a tool for economically improving the performance of existing facilities by addressing performance-limiting factors at individual MWWTPs in the areas of design, operation, maintenance and administration (WTC and Process Applications Inc., 1996; Daigger and Buttz, 1998). The CCP focuses on defining the optimum capability of existing facilities (USEPA, 1989a). Documented and demonstrated benefits of the CCP and associated optimization measures have included enhanced effluent quality (e.g., complete and consistent nitrification), improved operator motivation/confidence and, in some cases, reduced operating costs (WTC and Process Applications Inc., 1996).

The CCP uses a two-step process to diagnose and improve plant performance and effluent quality

**Figure 7.1 Steps Involved in Rerating, Debottlenecking and Optimization Projects (WERF, 1999)**


STEP 1: INITIAL ASSESSMENT
• WWTP INSPECTION
• DATA EVALUATION
• EFFLUENT REQUIREMENTS
• MASS BALANCE
• LOAD PROJECTIONS
• PERFORMANCE ASSESSMENT
• IDENTIFY CRITICAL PROCESSES
• DESIGN STEP 2
STEP 2: FIELD PROGRAM (POTENTIAL ELEMENTS)
• METER CALIBRATION
• PROCESS MONITORING
• STRESS TESTING
• PROCESS EVALUATION TECHNIQUES
• HYDRAULIC MODEL CALIBRATION
• PROCESS/SYSTEM MODEL CALIBRATION
STEP 3: ESTABLISH PROCESS/SYSTEM RATING
• ESTABLISH FUTURE LOADING/EFFLUENT SCENARIOS
• DEFINE ACCEPTABLE RISK
• MODELS/PROCESS EVALUATIONS ESTABLISH PROCESS/SYSTEM RATING
• DETERMINE BOTTLENECKS
STEP 4: CONCEPTUALIZE DEBOTTLENECKING AND OPTIMIZATION ALTERNATIVES
• THE CREATIVE PROCESS
• STATUS OF TECHNOLOGY
• PRELIMINARY SCREENING
STEP 5: ALTERNATIVE EVALUATION
• MODEL PREDICTIONS
• CAPACITY GAINS
• RISK ASSESSMENT
• CAPITAL COST ESTIMATION
• O&M PREDICTONS
• OTHER CONSIDERATIONS
STEP 6: IMPLEMENTATION
• SELECTION
• IMPLEMENTATION
• EVALUATION

(USEPA, 1989a; WTC and Process Applications Inc., 1996). The first step is a comprehensive performance evaluation (CPE) that examines four areas — operations, design, administration and maintenance — to identify performance-limiting factors. During the CPE, the major unit processes are evaluated to determine if they are capable of achieving the facility's discharge goals at existing flows and loads. If the CPE determines that the facility is potentially capable of achieving performance goals, the next step is to address the prioritized list of performance-limiting factors through a comprehensive technical assistance (CTA) program. If the facility is deemed incapable due to major design limitations, then the next step would be a design upgrade involving expansion of the existing facility or the construction of new replacement processes (USEPA, 1989a).

During the CTA, the prioritized performance-limiting factors are systematically addressed and resolved to enable the MWWTP to achieve the desired effluent quality. Operations, maintenance, administrative and minor design deficiencies are resolved through process control adjustments, optimization and minor facility modifications. This includes assistance to operations staff to ensure that the required process control skills are in place (e.g., through a process control workshop and/or site visits by the CCP facilitator for the provision of training and assistance). Assistance can be provided to resolve deficiencies in administration where these are identified.

"The Ontario Composite Correction Program Manual for Optimization of Sewage Treatment Plants" by WTC and Process Applications Inc. (1996) is available from the Ontario Ministry of the Environment and serves as a useful source document for optimizing the performance of an existing MWWTP. It represents a revised and expanded update of Chapters 1 to 3 of the U.S. EPA handbook "Retrofitting POTWs" (USEPA, 1989a). The manual describes methods to both evaluate an existing facility's capability for improved performance and achieve these improvements. It focuses on mechanical secondary treatment facilities (e.g., conventional activated sludge, extended aeration, contact stabilization, oxidation ditches, trickling filter, RBCs, etc.), including wastewater treatment and sludge handling aspects. Example CPE and CTA reports are provided in the appendices of the manual, including sample calculations, data collection forms and checklists.

### 7.1.5 Optimization Measures

The range of optimization technologies and techniques that are available to improve the performance of MWWTPs is large. The applicability of specific optimization measures for improved nitrification depends on various site-specific factors.

A listing of various potential optimization measures was provided in Chapter 6 (Table 6.1). Included under the optimization category in Table 6.1 are some of the techniques required during the initial plant evaluation phase (e.g., process audits and CCP) prior to the implementation of actual optimization measures.

The following are more specific optimization measures that were included in Table 6.1:

- enhanced process control
    - e.g., SRT, F:M ratio, RAS rate, biofilm excess growth, mixing, aeration, DO, pH, alkalinity, temperature, etc.;
- modified biological process configurations and/or modes of operation
    - e.g., step feed, SBR cycle times, on/off aeration, etc.;
- effective management of biological sludge inventory and characteristics;
- proper management of effluent and sludge recirculation rates;
- management of return streams to minimize impacts on the nitrification process;
- controlled management of biosolids, including thickening, stabilization, dewatering and disposal steps;
- utilization of on-line instrumentation and automatic process control;
- process modelling and simulation;
- pilot plant studies for the determination of optimum process configurations and modes and conditions of operation; and
- chemical addition
    - e.g., for influent BOD reduction, cBOD:TKN reduction and/or detoxification; pH adjustment; improved sludge settleability, etc.

In many cases, some of the measures listed under the plant retrofit category in Table 6.1 will be required in conjunction with plant optimization to permit an increased level of process control. Examples of such complementary liquid train retrofit measures include:

- aeration and/or oxygen transfer system replacement;
- secondary clarifier improvement;
- influent equalization;
- selector zone installation;
- reactor mixing system improvement;
- increased control of pumping rates; and
- I&C system improvements.

### 7.1.5.1 Enhanced Process Control

The development of effective process control and operational strategies is a key component of plant optimization. In the listing of specific optimization measures shown above, the category of enhanced process control reveals a number of process parameters applicable to nitrification systems. Many of these have been elaborated upon in Chapters 4 and 5 (Section 5.1.2). These process parameters are typically a main focus in optimization initiatives involving adjustments to operational strategies and process conditions for enhanced control (USEPA, 1993; WTC and Process Applications Inc., 1996; WEF, 1998). Some of the parameters relate to both suspended-growth and attached-growth processes, while others apply to one of the specific process types.

Since most unit processes at MWWTPs are coupled together, including recycle streams, consideration must be given to important process interactions that can affect treatment facilities. Therefore, process control strategies should consist of all approaches used to operate and optimize individual unit processes and to integrate these to maximize overall performance. This must include the controlled management of biosolids, which is a major area of concern in plant optimization.

Inadequate solids processing capacity is often the cause of poor performance and limited treatment capacity at MWWTPs. Limitations in solids processing (e.g., thickening, stabilization and/or dewatering operations) can impact the liquid train in the following ways:

- preventing sludge from being wasted from the liquid process as needed to maintain efficient treatment; and
- overloading the liquid train of the MWWTP by poor-quality recycle streams from overloaded and inefficient solids processing systems.

In addition to the proper management of sludge processing, reliable methods of ultimate disposal and/or reuse of biosolids must be available; otherwise, processed sludge will accumulate and limit plant capacity (Daigger and Buttz, 1998).

### 7.1.5.2 Operation and Maintenance

To ensure that desired process control strategies are effectively implemented, proper O&M procedures need to be practised and sufficient O&M resources must be allocated. The latter include items such as plant O&M budget, the number and skill level of plant staff and required facilities (e.g., laboratory and equipment). The full capability of a treatment plant cannot be realized without optimum O&M (Daigger and Buttz, 1998).

Computer software exists to assist with the planning and scheduling of operation and preventive maintenance tasks. Specialized software is available for the handling of laboratory data, inventory accounting and the preparation of computerized operations manuals. This can result in enhanced reliability of process control and maintenance. Furthermore, plant information can be accessed more easily and can be linked to larger management information systems (Daigger and Buttz, 1998).

Information related to the monitoring, control, O&M requirements and procedures related to activated sludge, trickling filter, RBC and dual biological processes are described in the Water Environment Federation's Manual of Practice (MOP) entitled "Operation of Municipal Wastewater Treatment Plants" (WEF, 1990). This MOP serves as a useful reference for operators of MWWTPs. It includes information related to process control systems for nitrifying activated sludge plants, including:

- methods, procedures and considerations related to RAS and WAS control;
- control and adjustment of other key process parameters, such as SRT, MLSS, DO, temperature, alkalinity and pH;
- operational aspects and procedures related to nitrification systems, including seasonal considerations;
- sampling, monitoring and analysis procedures; and
- planned maintenance programs for nitrification system secondary clarifiers.

When making process control adjustments, although a change in biological process operating conditions can be made quickly, the resultant effects on the biological system and sludge characteristics can take considerably longer to manifest themselves. A period of time of at least 2–3 SRTs should be allowed for a suspended-growth biological process response to an applied change in environmental conditions. If the desired change in sludge characteristics has not occurred in this time frame, additional pressure should be applied in a step-wise manner until improvements are established (WTC and Process Applications Inc., 1996).

### 7.1.5.3 Instrumentation and Control

An increased level of I&C represents an advanced MWWTP upgrade option that can result in increased treatment performance and process reliability. I&C can provide important information to the operator relating to the status of equipment and the measurement of selected parameters. This can facilitate the implementation of process control decisions as part of a plant optimization project (Daigger and Buttz, 1998). A balance between the use of a centralized monitoring and control system approach and the requirement for plant operators to maintain "hands-on" contact with the processes is recommended (Daigger and Buttz, 1998).

Software and hardware requirements for on-line monitoring and methods for developing a computerized data acquisition system are described in the process audit guidance manual by CG&S and Hydromantis, Inc. (1996). Common candidate parameters for on-line measurement include:

- flow rate;
- DO;
- pH and temperature;
- oxidation–reduction potential;
- suspended solids;
- turbidity;
- sludge blanket level; and
- nutrient concentrations.

A review of considerations for selecting and using on-line analytical instruments is contained in the "Wastewater Treatment Plant Instrumentation Handbook" (USEPA, 1985). On-line respirometry and automatic SRT control represent advanced technologies that are available to assist with biological process monitoring and control as part of process optimization. A brief overview of on-line process control parameters and related strategies for enhancement of the biological removal of total ammonia-nitrogen is also provided in Section 8.2.2.

## 7.2 Troubleshooting

Proper process monitoring, sampling, measurements, data compilation and analysis permit treatment plant performance assessments, troubleshooting and informed and effective control decisions (WEF, 1990). Testing and analysis can include the measurement of wastewater characteristics, microscopic examinations of process biomass and the measurement of sludge biological activity, such as specific nitrification rates. These procedures are described in the MOP "Operation of Municipal Wastewater Treatment Plants" (WEF, 1990). This can provide valuable information on microbial process health and can assist in process control and troubleshooting.

Prior to troubleshooting a problem, the treatment plant's capacity and limitations should be determined. Deficiencies in process design and equipment may limit performance or the ability to respond to problems (WEF, 1990).

### *7.2.1 Suspended-Growth Processes*

Some potential operational problems and considerations related to suspended-growth processes and possible corrective actions are covered briefly in this section.

A summary of common concerns related to design deficiencies and operational problems with SBR processes, including systems designed to achieve biological nutrient control, can be found in Hydromantis, Inc. and SBR Technologies Inc. (1998). Many of the reported operational concerns and recommended remedies related to process control apply to any type of activated sludge treatment plant.

#### 7.2.1.1 Sludge Characteristics

Sludge quality (i.e., physical and biological characteristics) is fundamentally an important aspect of successfully operating an activated sludge plant. A sludge that flocculates, settles and compacts properly will produce the best effluent that an activated sludge plant employing secondary clarification is capable of producing (Klopping and Melcer, 1999). This is important for nitrifying suspended-growth processes owing to the requirement for effective control of process SRT to ensure consistent performance. The ability to minimize effluent suspended solids losses and maintain increased MLSS concentrations and SRT levels allows nitrification to be accomplished at shorter HRTs. In the case of biological nutrient removal plants, additional factors can influence sludge quality and wastewater treatment efficiency due to interactions and competition between various microbial species.

Microbial process health can be revealed through sludge quality measurements, including the microscopic examination of the microbial species makeup of sludge. This information can reveal the factors causing sludge quality problems (WEF, 1990; Jenkins *et al.*, 1993; Daigger and Buttz, 1998; Klopping and Melcer, 1999), which can assist in process control and troubleshooting.

A poorly settling biomass (i.e., bulking sludge) can develop on occasion in activated sludge systems depending on process conditions. This is most often caused by the presence of excessive amounts of filamentous microorganisms, resulting in slowly settling and poorly compacting sludge (Daigger and Buttz, 1998). However, other factors, such as foaming, floating sludge and toxic shocks, can cause sludge losses (Appendix A). Consequently, the mixed liquor concentration and

SRT that can be maintained in the process may become limited, which can affect nitrification and wastewater treatment capacity and performance.

The presence of a particular filament type in mixed liquor as revealed through microscopic examination procedures can provide an indication of the prevailing environmental conditions and suggests remedial actions that can be taken. Microscopic analysis permits diagnosis of sludge bulking factors other than the presence of excessive quantities of filamentous organisms. It is generally possible to diagnose and develop a corrective action plan for many bulking problems (Jenkins *et al.*, 1993).

Problems with poorly settling sludge may be resolved through the adjustment of process operating conditions. Process modifications such as alternative operational modes (e.g., step feed) that lower the secondary clarifier solids loading rates and/or modified clarifier operations can be employed. Another option is to chlorinate the RAS stream to reduce the population of filamentous microorganisms relative to floc formers (Daigger and Buttz, 1998).

A possible process retrofit option is the addition of a biological selector zone upstream of the aeration tank to reduce the potential for excessive growth of filamentous organisms. Selectors can be classified as aerobic, anoxic or anaerobic, depending on the prevailing oxidation–reduction potential condition in this zone (WEF and ASCE, 1992). Another possible process upgrade option, albeit costly, is the replacement of the secondary clarification stage with membrane filtration for solids retention, which would eliminate activated sludge process reliance on sludge settling properties. The relative ease and cost of implementing the above-mentioned process retrofits will depend on the configuration of the existing treatment plant.

#### 7.2.1.2 Floating Sludge

The presence of brown clumps of sludge floating on the sedimentation clarifier surface may indicate denitrification activity in the secondary clarifier, causing nitrogen gas bubbles to be produced in stagnant sludge in the clarifier. Possible operational remedies can include modifications to the RAS rate and/or increasing the frequency of cleaning of clarifier walls and weirs.

#### 7.2.1.3 Sludge Wasting

The applied rate of sludge wasting can be used to control SRT, a key parameter in nitrifying activated sludge processes. Effluent suspended solids losses should be considered in the calculation and control of SRT levels.

Sludge wasting operations should be conducted carefully. Owing to the slow growth rate of nitrifying bacteria, sufficient time (i.e., at least 1 month) must be allowed to see the effect of changes in sludge wasting rates. Furthermore, accidental wasting of sludge can be disastrous because of the long time required for the system to recover, particularly at lower temperatures (WEF, 1990).

### 7.2.2 Attached-Growth Processes

Some potential operational problems and possible corrective actions related to the most common attached-growth processes (i.e., trickling filters and RBCs) are covered briefly below (WEF, 2000b). Detailed process characteristics and considerations relating to these and other attached-growth systems are provided in Appendix A.

#### 7.2.2.1 Trickling Filters

##### 7.2.2.1.1 Liquid Ponding and Excess Biofilm Growth

Liquid ponding may occur on top of the media surface and is associated with rock filters or incorrectly applied synthetic media. Ponding is caused by biomass clogging the void space of the media. Excessive biofilm growth and solids accumulation may occur in plastic media without visual ponding effects. The solids accumulations may cause blockage of air and liquid flow and reduce the effective aerobic surface area. A test can be run on the filter by stopping the distributor arm for 4–8 hours and analyzing the effluent TSS. If high levels of TSS are flushed out, then a solution would be to operate the influent distributor with higher SK values to favour routine sloughing of accumulated biomass.

#### 7.2.2.1.2 Biomass Sloughing and High Effluent Suspended Solids

Periodic cycles of high effluent suspended solids may occur due to uncontrolled sloughing of aerobic and anaerobic biofilm. Furthermore, organic overloading of the filters may result in the production of poorly flocculating suspended solids. Operation of the influent distributor at low speed with high SK values may permit more regular flushing of the filter biofilm media.

#### 7.2.2.1.3 Undesired Organisms

Some organisms, such as filter flies and midges, may develop on the biofilm media. These organisms and their larvae may be eliminated by flooding of the trickling filters, operating the influent distributor with high SK values to slough off the excess biomass growth and reduce the dry media areas, dosing of saline water and chlorination.

#### 7.2.2.1.4 Cold-Climate Applications

Trickling filter operation in cold-climate areas may lead to ice formation on the trickling filter due to heat losses to the atmosphere, high ventilation and recirculation ratio. Corrective actions may include filter cover installation, raising of wall height or installation of wind break on the walls, reduced and controlled ventilation and/or effluent recirculation with care to process impacts. Operation of the influent distributor with high SK values to slough off the excess biomass growth may be especially important in cold weather conditions where the solids tend to accumulate and reduce the aerobic surface area.

### 7.2.2.2 Rotating Biological Contactors

#### 7.2.2.2.1 Overloading

Excessive organic loads to the first passes of the RBC system may result in unbalanced biofilm growth, excessive shaft weight, oxygen transfer deficiencies, media clogging, biofilm anaerobic conditions, loping of air drive systems (i.e., non-uniform rotational speeds) and the growth of undesired organisms. Overloading may be controlled by more uniform flow distribution, influent step feed in the first passes of the train, removing the underflow baffles between passes to increase the media area of the first pass, increasing shaft speed to enhance oxygen transfer and biofilm sloughing, installing an air header for supplemental aeration or increasing the aeration rate.

#### 7.2.2.2.2 Excess Biofilm Growth

Biofilm control is required to avoid process impacts associated with overloading as mentioned above. Visual inspection of biofilm characteristics (e.g., thickness and colour) is required to monitor biomass development. Load cells for manual weighing of the shaft or electronic strain gauge load cells may be used to monitor shaft weight as a measure of biofilm development. In addition to the control measures mentioned for overloading, reduction of biofilm thickness may be achieved by periodically reversing the contactor shaft rotational direction, increasing shaft rotational speed, increasing air flow rate or isolating the RBC train with the excessive growth to starve the biomass. In severe cases, chemical cleaning of the media (e.g., with caustic) may be appropriate.

### *7.2.3 Nitrification Processes*

#### 7.2.3.1 Temperature Effects

Starting up and operating nitrifying systems at low temperatures are more difficult than conventional non-nitrifying secondary treatment processes (WEF, 1990). At wastewater temperatures below 10°C, starting up a system for nitrification or reestablishing nitrifying activity, once lost, may not be possible. However, nitrification activity can usually be maintained once developed, even at temperatures as low as 2–5°C.

Since the SRT required for nitrification is temperature dependent, seasonal adjustment of SRT through modifications in sludge wasting rate is often practised at suspended-growth MWWTPs in cold-climate applications. The modified SRT conditions should be applied well before the onset of cold temperature conditions to allow for the slow response time by the biological sludge.

#### 7.2.3.2 Effects of Industrial Effluent Contributions

Frequent and sudden changes in influent loadings to nitrifying processes can be a problem owing to the sensitivity of these microorganisms. Examples include discontinuous or seasonal industrial effluent discharges to the MWWTP or periodic additions of landfill leachate, where the associated loadings are significant and potentially toxic and inhibitory. Such problems with shock loads can be pronounced at smaller MWWTPs where industrial effluent contributions can be a significant fraction of the overall influent loading.

To minimize the impact of industrial wastes or leachates on nitrifying MWWTPs, especially in smaller applications, equalization of the problematical waste streams can be employed to reduce the variability and concentration of the potentially toxic loads contributing to the total plant influent. Furthermore, sewer use programs can be established requiring pretreatment of wastes at industrial sites, where necessary, prior to discharge to sewer. Evaluation and screening of contributing industrial effluents for potential toxicity using respirometric biotreatability testing techniques can be used to develop appropriate effluent management strategies (Schwing, 1999).

#### 7.2.3.3 Nitrification Inhibition and Ammonia Breakthrough

The following subsections describe some of the most commonly encountered problems with nitrification processes (USEPA, 1993).

##### 7.2.3.3.1 Microbial Toxicity

Microbial toxicity from inhibitory organic compounds or heavy metals may be encountered at MWWTPs, especially at those that receive a significant fraction of industrial effluent discharges and landfill leachate. Appendix B provides a list of some of the known inhibitory substances, including specific organic compounds and metals. Microbial toxicity can be detected by on-line DO monitoring (Section 8.2.2), where a sudden increase in DO concentration in the aeration tank may be associated with acutely toxic conditions. This DO concentration increase in the mixed liquor or process effluent may reflect a reduction of biological activity due to toxic effects. A significant accompanying increase in ammonia-nitrogen and decline in nitrate concentration in the effluent would further suggest the possibility of toxic inhibition.

Oxygen uptake rate measurements can provide valuable information on the possible presence of toxic substances affecting biomass activity (Section 8.2.2). Chronic toxicity, such as by the accumulation of heavy metals, may develop slowly and not be fully realized before 2–3 times the SRT. Microscopic examination of the biomass can provide an indication of process upsets.

#### 7.2.3.3.2 Ammonia Breakthrough

Total ammonia-nitrogen breakthrough in the effluent of a nitrifying MWWTP is generally associated with one or more of the following:

- inadequate oxygen transfer;
- inadequate alkalinity;
- excessive influent total ammonia-nitrogen concentration;
- biomass washout; and
- toxicity impacts.

Inadequate oxygen transfer may occur in suspended-growth and attached-growth reactors but is more frequently experienced in attached-growth systems. Suspended-growth and attached-growth nitrification processes must typically maintain minimum DO concentrations of 2 and 3 mg/L, respectively (Section 6.2.2.1). The bulk liquid DO concentration in the reactor is an indirect measure of the actual available oxygen within the biological floc or biofilm. Peak organic loads or high daily fluctuations may suppress nitrification activity, as the DO is used preferentially by the heterotrophic organisms for organic carbon oxidation. Thus, total ammonia-nitrogen may remain untreated or only partially oxidized.

Inadequate alkalinity and pH levels in a nitrification process may be caused by either low alkalinity levels in the raw wastewater or precipitation reactions during chemically assisted primary treatment (Section 5.3.2.2). This can lead to a severe pH depression, negatively affecting nitrification activity. Nitrifying organisms may acclimate to low pH conditions to some extent, between pH 6.0 and 6.5. Periodic low-pH inhibition may provoke the washout of the nitrifying organisms. Supplemental alkalinity adjustment may be required to avoid pH depression.

### 7.2.3.4 Suspended-Growth Systems

#### 7.2.3.4.1 Oxygen Requirement

Combined carbonaceous and nitrogenous oxidation systems increase the total oxygen demand due to the requirements of heterotrophic and autotrophic organisms. The existing type and configuration of aerators, the oxygen transfer rates and the aeration system capacities may not be sufficient to supply the oxygen required for overall treatment. Aeration facilities may need to be upgraded with increased blower capacity or retrofitting to more efficient air diffusers.

Clogging and ageing of some types of fine-pore diffusers may significantly reduce the oxygen transfer rate. Reduction of the oxygen transfer rate at the front end of the reactor, where the oxygen demand is highest, can lead to problems of inadequate DO further downstream in the aeration tanks. Consequently, nitrification efficiency may be significantly reduced. Mitigative measures can include an increase in air supply rate, retrofit of the aeration system with non-clogging fine- or coarse-bubble diffusers or provision of additional oxygen dissolution equipment at locations where the oxygen demand is highest in the aeration tanks. Alternatively, the influent load can be spread across the length of the aeration tanks (i.e., step feed mode).

#### 7.2.3.4.2 Bulking Sludge

The high SRTs used in combined carbon oxidation/nitrification activated sludge processes can result in the development of filamentous organisms, including the *Nocardia* genus. An excess of filamentous organisms may lead to bulking sludge events and scum generation. This scum should be managed to avoid entrapment in the liquid processing train. The points of entrapment should be equipped with covers so that scum can be seen and removed as required.

Strategies to reduce sludge bulking include the adjustment of process conditions such as operating in step feed mode, reducing sludge age, RAS stream chlorination, spraying chlorine solution directly on the scum and retrofitting the aeration tanks to add a front-end selector zone. Some of these strategies are discussed in other sections (e.g., Section 7.2.1), and further information may be found in WEF (1990), Jenkins *et al.* (1993) and Daigger and Buttz (1998).

### 7.2.3.5 Attached-Growth Systems

#### 7.2.3.5.1 Oxygen Transfer and Ammonium-Nitrogen Limitations

Autotrophic nitrification activity within an attached-growth system follows the heterotrophic carbon oxidation stage and will depend on the organic load applied to the system. The nitrogenous oxygen demand is also controlled by the quantity and specific activity of nitrifying organisms present in the process. The nitrifying population in the biofilm attached to the medium is limited by concurrent heterotrophic oxidation, oxygen transfer in the system and available ammonia.

Remedial strategies to promote the development and efficiency of a nitrifying population in the biofilm are outlined below:

- equalization of influent loads to reduce or avoid peaks in organic loading that can cause reductions in nitrification activity;
- recirculation of treated effluent within trickling filters to increase the wetting efficiency of the media and promote higher oxygen transfer rates;
- upgrading of the aeration system, including increased blower and fan capacities and supplemental air diffusers;
- periodic reversal of the influent flow path to the final nitrifying stages of the process, to promote the development of a more uniform nitrifying biofilm on each stage and to increase overall nitrification activity; and
- introduction of ammonia-laden sidestreams (e.g., from an anaerobic sludge digestion process) into the final nitrification stages to promote uniform biofilm growth and maximize nitrification activity in these zones.

#### 7.2.3.5.2 Biofilm Control

Attached-growth processes are affected by the biofilm thickness. Excess biofilm growth on the medium may significantly reduce oxygen, substrate and nutrient diffusion into the biofilm. The excess growth has the potential to block liquid and air flow, which may cause flow short-circuiting and negatively influence system performance. Thus, a constant controlled sloughing of the biomass is desirable to reduce the impacts of excess solids on biological process efficiency and to avoid uncontrollable high levels of solids in the effluent. Efficient biofilm control practices may also reduce the temperature dependency of attached-growth treatment systems (WEF, 2000b).

Control of excess biofilm can be achieved using various techniques according to the type of system. For trickling filters, biofilm sloughing may be achieved by controlling the influent and effluent recycle flow rates and intensity through an electric variable-frequency drive on the rotary distributor arm. The dosing intensity is represented by the SK factor, which is the depth of water deposited during the passage of one distributor arm (Appendix A). For RBCs, biofilm sloughing may be achieved by controlling the shaft drive and the air flow rate, if provided. Additional details on potential operational problems and corrective actions for trickling filters and RBCs are provided in Section 7.2.2. BAFs, fluidized-bed reactors and MBBRs may use their ability to create turbulence within the media volume, through filter backwash events or energy for media fluidization and suspension, to control the excess growth of biomass on the media.

### 7.2.3.6 Other Considerations

Nitrifiers are more sensitive to upset than most other microorganisms and require careful monitoring and process control. Particular attention should be paid to the important nitrification process parameters and operating conditions, as described in Section 5.1.2. The nitrification process is usually the key to successful operation of the entire MWWTP. When the nitrification process functions well, plant effluent is usually the best possible (WEF, 1990).

When undertaking operational adjustments (e.g., as part of corrective actions), consideration must be given to the interactions between the various process stages in the liquid and solids trains of the treatment facility. The operator needs a thorough knowledge of the biological nitrification process and how it fits into the overall plant operation (WEF, 1990).

As discussed in Section 7.1.2, when making adjustments to biological treatment process operating conditions, a period of at least 2–3 SRTs will be required for the full system response to take effect. Therefore, some of the desired process improvements may not be immediately evident following corrective actions. This applies especially to nitrification systems owing to the slow growth rate of nitrifying bacteria, particularly at lower temperatures. Any changes should be considered carefully.

## 7.3 Operator Training

The capability of MWWTP processes cannot be fully realized without optimum O&M. Adequate training should be provided to plant staff to enhance plant performance and ensure safety and compliance (Daigger and Buttz, 1998). All plant staff should be aware of the goals and objectives of the plant (e.g., discharge permit requirements).

Greater operational skills of treatment plant operators are generally required for the effective monitoring, control and troubleshooting of nitrification processes relative to the more conventional MWWTP processes employed for TSS and BOD control. This increased requirement applies in particular to plants that have been optimized to achieve higher treatment performance levels but may, as a result, be operating closer to their actual capacities.

Complex biological nutrient removal processes necessitate a higher degree of operator technical capabilities compared with the operation of conventional, fully aerobic plants. However, the necessary skills are easily learned and applied. A retraining program for the operators should be part of major retrofit projects, in particular in the case of biological nutrient removal processes (Randall *et al.*, 1992) or where novel technologies and equipment are installed. This should include theoretical and practical concepts (Hydromantis, Inc. and SBR Technologies Inc., 1998).

Training of plant operators on new technologies to ensure that systems are operated in the best manner possible should include instruction on:

- routine and non-routine O&M procedures;
- important analytical and process monitoring parameters and associated measurement procedures;
- interpretation of analytical and monitoring data; and
- implementation of appropriate process changes.

Training programs can be implemented by training consultants working together with plant staff. The consultant develops a training program with consideration to plant-specific processes and requirements, including information from technology vendors. The consultant may train the plant engineers and supervisory staff, who can then, in turn, train the plant operators. It is beneficial to include administrative, maintenance and operations workers in the training teams.

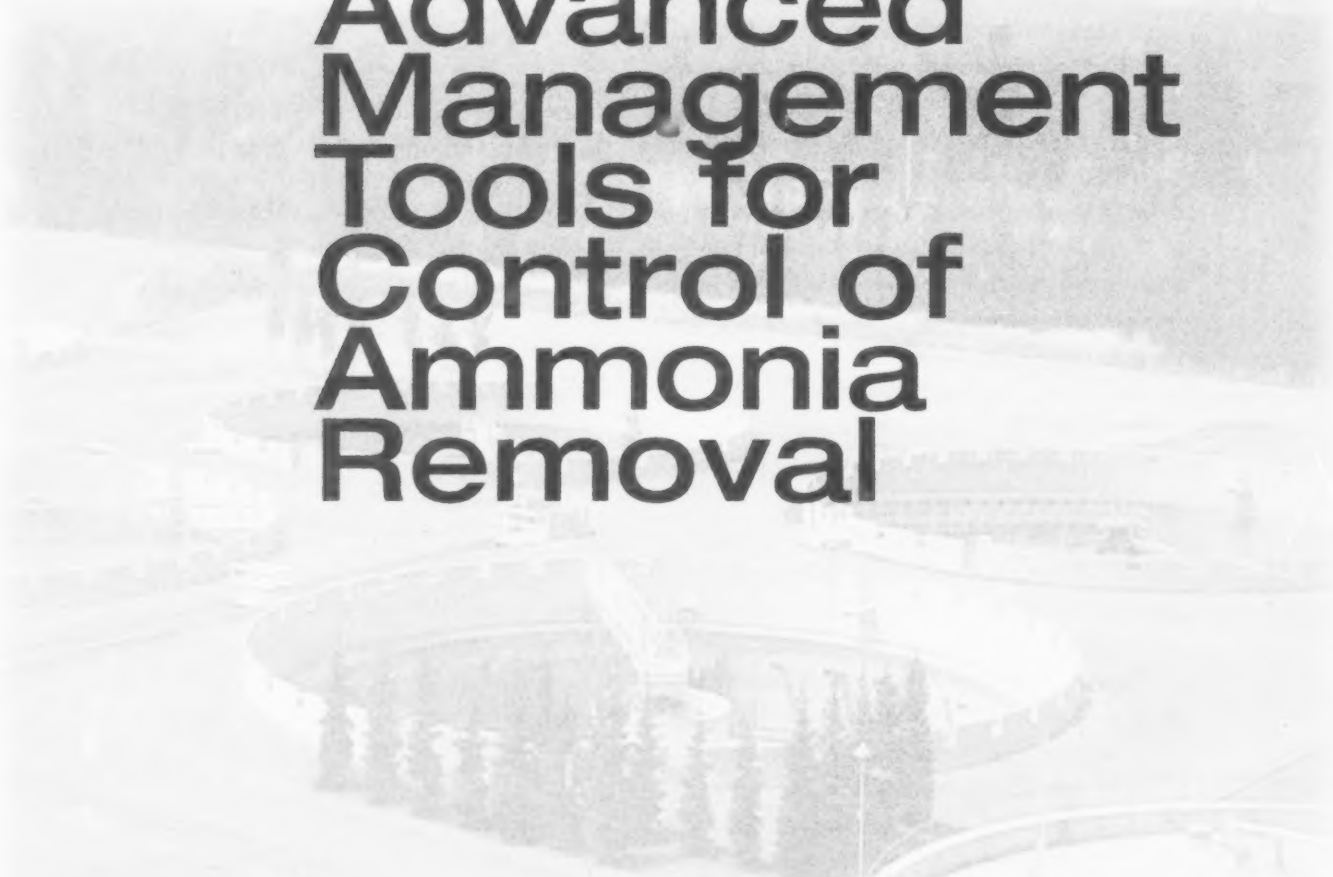
A major training objective related to the operation of nitrifying treatment systems is the development of acceptance by the operations staff of the lengthy periods of time required for the biological process to respond to process control adjustments. As a general reference, a time period of 3–5 times the applied SRT is necessary to establish important changes in biological sludge characteristics to reflect the new environment (WTC and Process Applications Inc., 1996).

### 7.3.1 Composite Correction Program

As part of the CCP, training of O&M staff and non-technical administrative personnel is included during the CTA program phase. This training can be provided by the external facilitator through:

- site visits, including the provision of on-site operator training according to scheduled training requirements as well as discussions with plant administrators at the start of the CTA; and
- telephone calls to monitor CTA progress and to train and encourage plant personnel concerning their responsibility for making critical plant observations, interpreting data and summarizing important indicators and conclusions.

# Chapter 8
# Advanced Management Tools for Control of Ammonia Removal



# Chapter 8
# Advanced Management Tools for Control of Ammonia Removal

Advanced management tools for the control of ammonia removal are available to engineers and managers/senior process staff to enhance and optimize treatment efficiencies at MWWTPs. Activated sludge and attached-growth biofilm process models have been developed over the past decades and incorporated into software packages. These packages may be used for the modelling of the biological process units and even the entire treatment plant. The treatment facilities may then be simulated under various scenarios to analyze the impact of upgrade alternatives. The software packages can be integrated with an on-line control system to monitor, regulate and optimize treatment plant process and operations management. Selected parameters can be monitored to enhance and control the biological ammonia removal process.

## 8.1 Model and Control Software

This section presents a brief overview of biological process models and wastewater treatment modelling and simulation software packages that are commercially available. Typical data requirements for model calibration and the types of investigation that can be addressed with modelling and simulation software are outlined. A short description of an advanced model-based control system for wastewater treatment facilities is provided.

### 8.1.1 Why Model?

MWWTP activated sludge systems increase in complexity with the expansion of their function from simple carbon removal to nitrification, denitrification and EBPR (WEF, 1998). Nutrient removal systems typically involve aerobic reactors with increased volume and/or aeration for nitrification purposes or a series of anaerobic, anoxic and aerobic reactors with internal mixed liquor recirculation and RAS.

Considering the complexity of such systems, mathematical modelling permits effective simulations of the behaviour of the existing plant and the upgrade alternatives to meet required performances. Modelling may be incorporated in treatment plant process audits (Section 6.3.2.1). Experimental testing and pilot plant studies can be conducted to corroborate and supplement the modelling activities (Section 6.3.2.2). Modelling and simulation of MWWTPs are used to identify cost-effective upgrade alternatives, operation and control and to facilitate the work of process designers and operators.

### 8.1.2 Models

Mechanistic models are the current approach in activated sludge treatment modelling. Biochemical reactions of the treatment processes occurring within the reactors are described in these models through mathematical expressions. Thus, the distinct biological processes and their interactions on the model variables are accounted for in the models.

Development of models for the processes used in municipal wastewater treatment has experienced rapid progress. The best-known model is the Activated Sludge Model No. 1 (ASM1), developed by the International Association on Water Pollution Research and Control (IAWPRC), which is now known as the International Association on Water Quality (IAWQ). In 1986, the IAWPRC released a report outlining a general model for the activated sludge process, which has been followed by many modifications and enhancements to allow prediction of several variables and additional physical, chemical and biochemical processes. The major models currently used are outlined as follows:

- *Activated Sludge Model No. 1*: ASM1 is limited to systems incorporating carbonaceous matter oxidation, nitrification and denitrification. EBPR is not included. Thus, two types of biomass are modelled: heterotrophic under aerobic or anoxic conditions for organic carbon substrate removal and autotrophic under aerobic conditions only oxidizing ammonium-nitrogen to nitrates for energy production. The ASM1 model has been shown to give a reliable description of the system response in wide ranges of system configurations (single and series reactor systems, aerated and non-aerated reactors and interreactor recycles), influent wastewater characteristics and operational parameters (WEF, 1998). Further information on this model can be found in Henze *et al.* (1987a,b).

- *Activated Sludge Model No. 2*: ASM2 is an extension of the first model, primarily to handle EBPR systems. This model includes mathematical process expressions describing chemical phosphorus removal. The processes described by the model include those associated with hydrolysis, heterotrophic and autotrophic biomass and phosphorus-accumulating bacteria (EBPR). The hydrolysis processes consist of the degradation of slowly biodegradable organic material into soluble substrate under aerobic, anoxic and anaerobic conditions. The heterotrophic and autotrophic processes are similar to those in ASM1. The phosphorus-accumulating bacteria processes are as per the description provided in Section 5.3.2.1, and the fermentation of organic material under anaerobic conditions is accounted for in the model. The kinetic parameters of the model are automatically adjusted according to the temperature variation. Further information on this model can be found in Henze *et al.* (1994).

- *Activated Sludge Model No. 2d*: ASM2d is a minor extension of the ASM2 model. Additional processes are included to account for the phosphorus-accumulating bacteria that can use stored organic polymers for denitrification in an anoxic environment. Further information on this model can be found in Henze *et al.* (1998).

- *Mantis Model*: The Mantis Model is identical to the ASM1 with three basic exceptions. The first is that uptake of nitrate as a nutrient source by biomass can occur when there is a lack of ammonium-nitrogen in the environment. The second is that kinetic parameters of the model are temperature dependent, and the third is the inclusion of aerobic denitrification potential in the model. Further information on this model can be found in Hydromantis, Inc. (1999).

- *General Model*: The General Model is a dynamic model for biological nutrient removal activated sludge systems. The model incorporates carbon oxidation, nitrification, denitrification and biological excess phosphorus removal behaviour (WEF, 1998). This model was developed by Dold (1990) and is derived from ASM1 and the Wentzel *et al.* (1989) model for phosphorus-accumulating organisms. The model accounts for nitrate uptake as a potential nutrient and anoxic growth of phosphorus-accumulating organisms. The model is termed general as it can be applied directly for the simulation of aerobic, anoxic-aerobic or anaerobic-anoxic-aerobic systems (Hydromantis, Inc., 1999). The kinetic parameters of the model are temperature dependent.

- *Steady-State Model*: The Steady-State Model has been developed for nitrification-denitrification EBPR systems, and the main function of the model is to predict the amount of phosphorus removal and the total mass of volatile solids in a nitrification-denitrification EBPR system operated at a given sludge age and with a specified anaerobic sludge mass fraction. Further information on this model can be found in Wentzel *et al.* (1990).

Other models have been developed for sedimentation, including clarification and thickening (Tackàs *et al.*, 1991), biofilm, anaerobic digestion and disinfection (WEF, 1998). Thus, it is possible to achieve modelling of the complete treatment plant facilities, including all process units and waterworks.

### 8.1.3 Commercially Available Software Packages

With the advent of powerful desktop computers that can run mathematically complex models, modelling and simulation software packages specifically designed for wastewater treatment facilities are now available. These packages typically include predeveloped models of the process units of a treatment plant facility or even an entire plant. Some of these software packages include several process unit models that represent each unit process/operation (from headworks to disinfection) that may be used as building blocks to develop a model according to the existing or foreseen treatment plant facilities and operation. Other software applications do not permit such model development but instead incorporate prebuilt plants that must be configured to model the treatment plant. Some packages incorporate graphical user interfaces, process model libraries and other productivity tools, such as sensitivity analysis and mathematical optimization modules (CG&S and Hydromantis, Inc. 1996).

A brief list of commercially available modelling and simulation software packages is provided in Table 8.1. The selection of a software package as a tool to achieve modelling and simulation efforts must consider the requirement for modelling of either a single unit process or the entire treatment facility. The capability to develop a model for the complete treatment plant with each of the process units incorporated in an integrated model is provided by only some of the listed software packages (CG&S and Hydromantis, Inc. 1996). Modelling development is an ongoing and dynamic sector, and new versions with improved models, speed graphics display, range of processes and ease of use will continue to be developed.

The listing in Table 8.1 is not exhaustive, and other products may be available. The objective is to provide information on modelling and simulation tools for assistance in MWWTP design, optimization and control.

### 8.1.4 Model Development and Data Requirements

The biological models described are the basic tools that permit effective simulations of processes and biomass characteristics under steady-state or dynamic conditions. The first step into simulation is to select the model to be used according to the type of existing or foreseen biological reactor and the process level to be achieved — carbon removal, nitrification, denitrification or EBPR. The preparation of a calibrated model for a full treatment plant or a single process unit may be achieved using predefined models of the process units, such as those available with the software tools listed in Table 8.1. The model provides a representation of the real system. Thus, the model calibration effort is an important aspect to ensure a good approximation of the treatment plant facilities.

Model development is typically completed in three steps (CG&S and Hydromantis, Inc., 1996). The first step is the preliminary analysis of the treatment plant layout and the raw influent data. The second step is the dynamic model construction using the data collected in the first

**Table 8.1 Major Modelling and Simulation Software Packages for MWWTPs (CG&S and Hydromantis, Inc., 1996; WEF and ASCE, 1998)**

| Software Name | Notes | Supplier |
|---|---|---|
| AQUASIM | Multiple models, includes open-channel models | EAWAG<br>Ueberlandstrasse 133<br>CH-8600 Dubendorf,<br>Switzerland |
| ASIM | User definition of biokinetic models, version 3.0 includes the ability to use ASM1, ASM2 and other biological system models, steady-state and dynamic simulations are possible | EAWAG<br>Ueberlandstrasse 133<br>CH-8600 Dubendorf,<br>Switzerland |
| BioWin | Basic model library, modelling and simulation of activated sludge process and several other wastewater treatment processes, ability to use ASM1 extended to include biological phosphorus removal, PC platform, steady-state and dynamic simulations are possible | Envirosim Associates, Ltd.<br>482 Anthony Drive<br>Oakville, Ontario L6J 2K5 |
| GPS-X | Large process model library, development of full-plant models, modelling and simulations of activated sludge and complete wastewater treatment plant units processes, models available for primary and secondary clarification, flotation, attached-growth (trickling filter, RBC, BAF), model development, productivity tools, PC platform, steady-state and dynamic simulations are possible | Hydromantis, Inc.<br>1685 Main Street West,<br>Suite 302<br>Hamilton, Ontario L8S 1G5 |
| SSSP | Simulation of single-sludge processes, basic IAWQ biokinetic model (ASM1), steady-state and dynamic simulations are possible | C.P.L. Grady<br>Clemson University<br>Clemson Research Park<br>Clemson, SC 29634-0919,<br>USA |
| STOAT | Development of full treatment plant simulation models and includes activated sludge unit process and others, PC platform | Water Research Center plc<br>Frankland Road, Blagrove<br>Swindon, Wiltshire, U.K.<br>5N5 8YF |

step and the various process unit models available in the modelling software as building blocks. The last step is to calibrate the treatment plant model under steady-state and dynamic conditions. Verification of the calibrated model is then conducted by comparing simulation results from the model with data from the plant. Simulation of the treatment plant under various scenarios for upgrade alternative selection, plant operation and control may then be conducted.

Modelling of a wastewater treatment plant requires assessing several parameters related to the treatment plant itself and to the models upon which the calibration of the plant is achieved. Many of these parameters can be measured directly, while others are based on experimental data from the literature. The parameters that cannot be measured directly or are derived from the literature are estimated with modelling dynamic sensitivity analysis and optimization modules, based on treatment plant process performance records and/or field/laboratory experimental data. The quality of a model is strongly influenced by the quality of the data used in the calibration step.

In general, the data requirements for model calibration include the following categories:

- Influent characteristics
  - basic parameters of raw wastewater, including organic material biodegradable and non-biodegradable subfractions, nitrogenous material with ammonia/ammonium and TKN biodegradable and non-biodegradable subfractions, TSS, VSS, and total and soluble phosphorus;
  - composition of auxiliary input streams;
  - organic material preferably expressed in terms of COD, as it provides a consistent basis for description of the activated sludge process.
- Physical plant data, including:
  - configuration of the treatment plant, including process mechanical equipment capacities;
  - dimensions of the unit processes;
  - process flow scheme and hydraulic characteristics (flow lines, recycle lines, bypasses, true hydraulic characteristics, plug flow, completely stirred tank reactors, etc.);
  - sludge collection and withdrawal process.
- Operational plant data
  - current hydraulic flow scheme, including bypasses, flow splits, overflows, combiners, sludge recycle, sludge wastage rate, etc.;
  - unit process influent and effluent flows and compositions;
  - aeration tank and clarifiers process operational data, such as MLSS, MLVSS, DO, oxygen uptake rate (OUR), etc.;
  - sludge blanket height and underflow solids concentration;
  - chemical dosages.
- Kinetic and stoichiometric model parameters
  - organic, nitrogenous and phosphorus material removal parameters;
  - primary and secondary clarifiers settling parameters.

Further details on influent characteristic requirements for modelling, field or experimental measurements and estimation may be obtained in many references, such as Water Research Commission (1984), Henze *et al.* (1987a), CG&S and Hydromantis, Inc. (1996) and WEF (1998). Information on kinetic parameters for the removal of organic and nitrogenous material may be found in Henze *et al.* (1987a) and Ekama *et al.* (1986). Default values for these kinetic parameters are provided in most of the software listed in Table 8.1 and may serve as a starting point, as many of these parameters do not change appreciably for different systems, with the exception of nitrifier growth rate (Section 6.3.2.2). Further information on settling model parameters can be found in Tackàs *et al.* (1991).

### 8.1.5 Modelling and Simulation Benefits

Simulations of the treatment plant may be used to assess various upgrade alternatives, plant operation and control scenarios, and associated benefits. These simulations could be carried out in conjunction with process audits (Section 6.3.2.1) or in the course of regular operation to assist the operators with treatment plant control and management. Several investigations and related advantages may be achieved and obtained with the treatment plant model simulations and are outlined as follows (CG&S and Hydromantis, Inc., 1996):

- reactor configurations for nutrient removal (nitrification, denitrification and EBPR); simulations to assess flow rates, wastewater feeding pattern, internal mixed liquor recycles, reactor volume requirements (HRT) and reactor environmental conditions (aerobic, anoxic, anaerobic) to achieve nutrient effluent criteria;

- treatment plant and individual process capacity evaluation;
- bottleneck identification of the process units;
- hydraulic load change impact evaluation on plant efficiency;
- hydraulic bypass impact reduction;
- management of wet-weather flow, combination of bypass fraction and step feed;
- aeration system optimization, including oxygen transfer efficiency upgrade and energy-saving strategies;
- sludge recycle and wastage strategies optimization to control MLSS and SRT; and
- sludge production evaluation.

### 8.1.6 Advanced Model-Based Control System

The development of a dynamic model for a wastewater treatment plant can be used as the centrepiece of a comprehensive control system. Thus, an integrated computer-based approach to treatment plant operation and control incorporates a calibrated dynamic model connected to a plant SCADA database system by means of a software link. A user-friendly operator interface for operation management is required with the ability to adjust parameters and unit operation on-screen. The operator interface is allied to the dynamic model located in the background and invisible in current operation.

The SCADA layer of the system typically includes a historical database and linkages to the sensors and control actuators. This layer provides automatic on-line and off-line selected plant operating parameters to the computer control system supporting the dynamic model. The measured variables then feed the treatment plant site-specific model. The computer control system treats the data and accordingly sends operator-approved control actions to the plant via the SCADA system.

The implementation of an advanced model-based control system provides significant advantages in terms of process monitoring and control, cost-effective optimization and operation, and treatment plant operation management.
The major benefits are as follows:

- process optimization in real time;
- effluent quality prediction over the next few hours or over longer time scales, providing the operators with time to react and prevent excursion outside the operational range;
- advanced process warnings requiring operator intervention and enhanced rapidity of action;
- ability to extend and maintain the dynamic model calibration with on-line running optimization modules;
- facilitation of day-to-day treatment plant operation;
- off-line mode simulation to analyze the impact of operational strategy modifications for treatment plant optimization and operation management during maintenance periods, such as the bypass of an aerated basin or a secondary clarifier;
- cost-effective operation enhancement in terms of energy and chemical consumption;
- troubleshooting of treatment plant or single unit process;
- investigation of what-if scenarios;
- sensor fault detection warning; and
- training of managers and operators.

## 8.2 Process Control Parameters

Process control parameters and related strategies should be able to maintain high levels of treatment performance under a wide range of operating conditions. Process control should be consistent with the level of treatment requirements and be developed in conjunction with treatment plant managers and operators. I&C systems must provide sufficient flexibility to allow operators to select the control methods that best meet their goals and objectives (WEF and ASCE, 1998). Greater process stability, consistency in effluent quality and reduced operating costs are typically obtained with the installation of on-line monitoring and control systems. Quick response to sudden condition changes, such as total ammonia-nitrogen load, can decrease the risk of not meeting the effluent requirements (Thornberg *et al.*, 1993). This section presents an overview of typical MWWTP liquid train monitoring and process control parameters.

### 8.2.1 Wastewater Treatment Plant Monitoring

Common process parameters that are typically monitored and measured in an MWWTP liquid train are summarized in Table 8.2. The parameters and flow rates indicated are provided as an example and must be adjusted for the type of MWWTP and degree of complexity and treatment requirements. Measurement of primary effluent characteristics is critical to characterize a biological system and should not be underestimated.

The outlined parameters may be obtained through manual or automatic composite samples and be analyzed in the MWWTP laboratory facilities and/or external laboratory. Sidestreams from sludge handling and WAS streams may be periodically operated, and manual samples may have to be taken. Manual grab samples represent a snapshot, while composite samples represent an average over a period of time. Real-time data that represent the dynamic behaviour of the plant may be obtained with the use of on-line monitoring. On-line monitoring utilizes temporarily or permanently installed instrumentation. The parameters that can be measured with on-line monitoring include flow rates, air flow rates, DO, suspended solids, nutrients (phosphorus, ammonium-nitrogen and nitrate), temperature and pH, oxidation–reduction potential and sludge blanket level. Detailed information on flow rate metering devices, probes, meters, nutrient analyzers (including ion-specific electrode) and automatic sampling equipment and procedures are available in many references, including CG&S and Hydromantis, Inc. (1996) and APHA *et al.* (1998).

### 8.2.2 Process Control Parameters and Related Strategies

A variety of process control parameters and related strategies are used for the control of ammonia removal. Process measurements provide the operator with information on which to base judgements and allowing problems to be anticipated and resolved. Overall control strategy may require improvements in the plant's hardware, including on-line monitoring instrumentation. In such a case, real-time process data and flow rates are collected by a data acquisition system and further analyzed for control requirement. Simultaneous collection of real-time process parameter data allows identification of various process interactions. Following the identification of the process interactions, the treatment plant control and operation can be adjusted to optimize the plant performance and meet the effluent requirement (Daigger and Buttz, 1998).

Process parameter analysis within each of the secondary treatment reactors in series or within each period of a treatment cycle (such as with SBR) favour understanding of the process

**Table 8.2 Common Process Parameters and Flow Rates Measured in an MWWTP Liquid Train**

| Raw Influent and Sidestreams | Primary Effluent | Secondary Treatment | Secondary Clarification | Final Effluent |
|---|---|---|---|---|
| **Most Common Parameters** | | | | |
| $BOD_5$ | $cBOD_5$ | MLSS | RAS/WAS suspended solids concentration | $cBOD_5$ |
| $cBOD_5$ | TSS | MLVSS | Sludge blanket height | TSS |
| TSS | Total phosphorus | DO | | Total phosphorus |
| Total phosphorus | Soluble phosphorus | | | Soluble phosphorus |
| Soluble phosphorus | TKN | | | TKN |
| TKN | $NH_4^+$<br>$NO_3^-/NO_2^-$ | | | $NH_4^+$<br>$NO_3^-/NO_2^-$<br>Temperature pH |
| **Process Flow Rates** | | | | |
| Influent | | | RAS<br>WAS | |
| **Less Common Parameters** | | | | |
| Soluble $BOD_5$ | Soluble $BOD_5$ | Soluble COD | | Soluble $BOD_5$ |
| $NH_4^+$ | COD | $NH_4^+$ | | COD |
| $NO_3^-/NO_2^-$ | Alkalinity | | | |
| COD | | Soluble phosphorus | | |
| | | Alkalinity | | |
| | | pH | | |
| **Process Flow Rates** | | | | |
| | | Process air | | Effluent |

reactions in the biological treatment of the plant and facilitate the process optimization and control.

The major process control parameters and related strategies are outlined in the following subsections.

#### 8.2.2.1 Temperature

Wastewater temperature follow-up permits the adjustment of the treatment plant controlling factors including SRT, WAS and RAS rate, and biofilm control techniques to maintain the required biomass in place for nitrification process efficiency (Section 5.1.2). The RAS rate is important in maintaining the MLSS concentration, and the WAS rate is important in controlling the SRT or sludge age. Automatic control of SRT in suspended-growth systems can be implemented.

#### 8.2.2.2 pH

The pH measurement provides information on the wastewater buffering capacity efficiency and the alkalinity adjustment requirement.

A reduction of the pH to below the optimum operating range (Chapter 4) indicates a possible lack of alkalinity. Severe pH reduction may reduce or halt nitrification process activity.

### 8.2.2.3 Alkalinity

Primary-treated wastewater alkalinity measurement ensures that sufficient buffering capacity is provided to satisfy the nitrification process needs and to maintain a minimal alkalinity residual in the final effluent. Alkalinity measurement within biological reactors is used as an indicator of the nitrification process activity, as alkalinity is consumed through the nitrification reaction. MWWTP cases that involve raw wastewater with low alkalinity or primary chemical precipitation with a significant reduction of alkalinity (Section 5.3.2.2) may require supplemental alkalinity adjustment.

### 8.2.2.4 Dissolved Oxygen

DO measurements within biological reactors allow a number of issues to be controlled and addressed, outlined as follows:

- Monitor that sufficient DO concentration is maintained within the reactors to favour nitrification process activity. Suspended-growth and attached-growth aerobic reactors typically maintain a minimum of 2 and 3 mg DO/L, respectively, for nitrification (Section 6.2.2.1).
- Control the oxygen supplied and the aeration system operation. The oxygen supplied to the biological reactors should ideally meet the oxygen demand from the organic carbon oxidation and the nitrification process. The control of oxygen supply with the DO measurement is typically achieved with the following strategies:
    - A preprogrammed, time-based pattern that approximates typical carbonaceous and nitrogenous oxygen demand variations exhibited by daily fluctuations in influent flows and loads is developed from historical records of monitored DO (WEF and ASCE, 1998). Thus, the oxygen supplied by the aeration system is adjusted during periods of the day according to historical mixed liquor DO and influent load variations. The time-based pattern should be periodically evaluated and revised as necessary. Time-based DO control is recommended as a backup to more complex control strategies.
    - Set-point DO concentration is used to automatically control the oxygen supply input according to on-line DO and air flow rate measurements (WEF and ASCE, 1998). Thus, during periods of low influent loads, the DO can be maintained at the minimum set point; during periods of high influent loads, the DO will tend towards the maximum set point (Puznava *et al.*, 1999). The oxygen supply and the air flow rate are then adjusted according to the influent carbonaceous and nitrogenous load fluctuations that are reflected in the on-line DO variations. This strategy can significantly reduce excess oxygen supplied during low influent loading periods.

In general, it can be observed that as long as the load follows a regular pattern, a well-adjusted timetable is as good as a set-point DO control; however, on days of load deviations from normal fluctuations, which can occur often in most plants, the automated set-point DO control can provide significant energy savings for aeration (CG&S and Hydromantis, Inc., 1996; Puznava *et al.*, 1999).

Air flow rate adjustment associated with the oxygen supply control strategies may proceed with various equipment arrangements, outlined as follows:

  - Two-speed motor blowers control providing two fixed air flow rates. This option is used with the preprogrammed time-based pattern.
  - Variable-frequency drives with positive displacement blowers and air flow inlet modulating valves with centrifugal blowers are used to adjust the air flow rate requirements. The variable-frequency drives regulate the motor speed of the positive displacement blowers to provide the required air flow rates according to the set-point DO concentration strategy. The air flow inlet modulating valve positions with the centrifugal blowers vary to provide the required air flow rates according to the set-point DO concentration strategy.
- Detect toxic discharges from the biological reactor process upsets. A sudden increase of on-line DO concentration monitoring in the aeration tanks could be associated with a toxic discharge to the treatment plant facility (CG&S and Hydromantis, Inc., 1996). This increase of DO concentration reflects the reduction of biological activity due to the toxic compound. Inhibition of nitrifiers may occur by a number of inorganic and organic chemicals and metals (Chapter 4).

### 8.2.2.5 Ammonium-Nitrogen

On-line measurement of ammonium-nitrogen is used as an indicator of the efficiency of the nitrification process. Ammonium-nitrogen measurements permit several issues to be controlled and addressed, outlined as follows:

- Control the oxygen supplied to achieve nitrification with high and low effluent ammonia set points. Diurnal increases of on-line monitored effluent ammonium concentration have been shown to be related to influent diurnal loading variation. Similarly, high effluent ammonium concentration were linked to low on-line DO levels. Thus, an air flow rate control based on on-line DO set points in the biological reactors overruled by high and low effluent on-line ammonium set points may be used to optimize the oxygen supplied (CG&S and Hydromantis, Inc., 1996; Puznava *et al.*, 1999; Sorensen *et al.*, 1999). The DO set point is determined directly from the effluent total ammonia-nitrogen concentration. This control strategy provides the ability to vary the air flow rate and the oxygen supply input as a function of the ammonia loading variation (WEF and ASCE, 1998). This strategy can benefit effluent compliance for total ammonia-nitrogen (Puznava *et al.*, 1999).
- Control of the treatment phase lengths. On-line measurement of ammonia in the bioreactors provides the information for dynamic control of the treatment phase lengths (Thornberg *et al.*, 1993). The control strategy is based on the principle that a nitrification phase ends when total ammonia-nitrogen is low. During the nitrification process, the ammonium concentration is monitored, and the phase is ended by comparing with effluent ammonia set points. This strategy may be coupled with control of the oxygen supplied to achieve nitrification with high and low effluent ammonia set points. Thus, the air flow rate and the DO set point are modulated as a function of the total ammonia-nitrogen load variation.

  This strategy for the control of treatment phase lengths has been utilized in certain continuous-flow oxidation ditch processes with denitrification, including two-ditch and three-ditch systems, such as BioDenitro DE and BioDenitro T, respectively (Appendix A). The Superior Tuning and Reporting (STAR) software package manufactured by Krüger includes these control strategies for control of phase lengths and oxygen supply for enhanced nitrification and nutrient removal. SBR technology could benefit from this type of control strategy to adjust the duration of the aerobic treatment cycles.

- Equalize the ammonia-laden sidestreams from sludge handling processes in a buffer tank and the peak raw influent loads in existing emergency storage ponds. The waters are then dosed back into the biological treatment by using a combination of a timetable and an on-line ammonia set-point control (CG&S and Hydromantis, Inc., 1996; Sorensen *et al.*, 1999).
- Control the number of active and standby biofiltration cells. This strategy is applied to aerobic submerged fixed-bed reactors (BAF), particularly the Biostyr process (Sorensen *et al.*, 1999). The function of this control strategy is to select the number of biofiltration cells in a standby mode based on the influent flow rate and the influent/effluent total ammonia-nitrogen concentration. Low flow and low total ammonia-nitrogen concentration increase the number of cells on standby. Thus, a nearly constant high total ammonia-nitrogen load is kept on the biofiltration cells, which provides higher efficiencies on each cell. Furthermore, the operation of an appropriate number of biofilters enables the development of a high level of active biomass, which can treat high total ammonia-nitrogen loads as well as more usual loads (Sicard *et al.*, 1999).
- Control high total ammonia nitrogen peak loads to increase the reactor aerated volume. This strategy is applied with the Biostyr reactor (Sorensen *et al.*, 1999). The effluent total ammonia-nitrogen concentration is measured on-line and compared with a high ammonia set point. If the effluent total ammonia-nitrogen exceeds the high set point, the process air intake automatically switches from middle media aeration to bottom aeration (Section 5.1.4). This control strategy permits a near doubling of the aerated volume.

#### 8.2.2.6 Oxygen Uptake Rate (OUR)

OUR is recognized as a means for monitoring and control of the activated sludge process. The OUR is defined as the rate at which the biomass microorganisms are using oxygen for the consumption of carbonaceous and nitrogenous compounds and their growth. Thus, OUR represents a measure of the biological activity. The OUR is usually expressed as milligrams of oxygen per gram MLVSS per hour, which is also referred to as the respiration rate. High OURs indicate high biological activity, and low OURs indicate low biological activity (Metcalf & Eddy, 1991). The OUR test description and the equipment required are presented in CG&S and Hydromantis, Inc. (1996) and APHA *et al.* (1998). The OUR or actual respiration rate in the aeration tank can be measured with commercial respirometers if the sample streams of wastewater and activated sludge are mixed in the respiration chamber in such a way that the ratio of wastewater flow and chamber volume equals the ratio of influent flow and aeration tank volume (Hydromantis, Inc., 1996).

Respirometry-based techniques may be used for preliminary biotreatability testing (Section 6.3.2.2) and evaluation of modelling parameters.

OUR measurements permit the control of several issues, outlined as follows:

- Indicate toxic effects on activated sludge. Toxic compounds in wastewater can drastically reduce the biological activity and consequently the actual respiration rate. However, fluctuations in wastewater loading applied to the activated sludge also provoke variation of the actual respiration rate. Thus, a variable, such as the maximum respiration rate, that is less sensitive to the applied loading strength must be used to differentiate the toxic input effect.

The maximum respiration rate is obtained by loading the respirometer with an excess of raw wastewater. The substrate concentration becomes high, and the maximum respiration rate is approached. This rate is then used as the reference rate without toxic input. Thus, toxic compounds discharged in the raw wastewater will also be applied in excess to the respirometer unit. The toxic effect will be manifested by a reduction of the maximum respiration rate. As the biomass exposure to toxic compounds is higher in the respirometer than in the treatment plant aeration tanks, toxic input effect on the biomass can be detected before inhibition of the biomass and used as an early warning by the operators (Hydromantis, Inc., 1996).

- Control the treatment phase lengths. The on-line OUR measurement of mixed liquor can be used for the determination of carbon, ammonium and nitrite oxidation rates. An on-line batch respirometer with selective addition of nitrification inhibitors is used to rapidly evaluate and monitor these various oxidation rates and, more specifically, the nitrification process efficiency in activated sludge tanks (Surmacz-Gorska et al., 1996).

This control strategy permits the detection of ammonium oxidation to nitrite during the aerobic phase. The decline of ammonium and nitrite oxidation rates coincides well with the decline in ammonium and nitrite concentrations. When the ammonium oxidation rate drops to a very low level, ammonium has been completely oxidized to nitrite. Similarly, when the nitrite oxidation rate declines, nitrite has been oxidized to nitrate. The aeration treatment phase length could then be adjusted according to the oxidation rate variation. Thus, the aeration phase could be stopped or the oxygen supplied reduced when the ammonium oxidation rate indicates that all ammonium has been converted to nitrite.

Considering the daily variations of influent loads, this method permits the control of the aerobic treatment phase lengths to cope with the load fluctuations. In an SBR application, this control strategy translates into longer cycles with high influent loads and shorter cycles with low influent loads.

#### 8.2.2.7 Oxidation–Reduction Potential

The oxidation–reduction potential (ORP), also referred to as the redox potential, measures the prevailing environmental oxidative conditions in mixed liquor. The need for enhanced nitrogen removal has emphasized the requirement for aerobic and anoxic zones. ORP levels can be related to environmental treatment conditions and linked to the nitrogen removal reactions, i.e., nitrification/denitrification, as shown in Table 8.3 (Charpentier *et al.*, 1986). ORP levels are used by some manufacturers to provide further distinction between aerobic carbon removal and nitrification. ORP control is typically applied to low-load activated sludge reactors with high nitrogen quality effluent requirements (Charpentier *et al.*, 1998).

A high ORP limit of 160 mV (redox Ag/AgCl electrode type) for the nitrification process has been used in aeration tanks to obtain an effluent ammonium concentration lower than 3 mg/L (Charpentier *et al.*, 1986). The optimized nitrification process has been identified as 200 mV for the upper threshold of ORP regulation setting (Ag/AgCl electrode) (Charpentier *et al.*, 1998). The nitrification process could potentially be controlled with on-line ORP set points overruled by high and low effluent on-line ammonium set points. Thus, according to the effluent ammonium

## Table 8.3 Oxidation-Reduction Potential and Removal of Carbon and Nitrogen Compounds (Charpentier *et al.*, 1986)

| REDOX POTENTIAL refer.:Ag/Ag Cl | TREATMENT CONDITIONS | C | N |
|---|---|---|---|
| 100 mV | Presence of dissolved $0_2$<br>Aerobic zone<br>Aerobic respiration | OXIDATIVE PATHWAY<br>→ $CO_2 + H_2O$ | NITRIFICATION<br>$NH_4^+ \rightarrow NO_3^-$ |
| 0 mV+ | Presence of $NO_3^-$<br>Absence of dissolved $0_2$<br>Anoxic zone<br>Anaerobic respiration | OXIDATIVE PATHWAY<br>→ $CO_2 + H_2O$ | DENITRIFICATION<br>↑<br>$NO_3^- \rightarrow N_2$ |
| - 300 mV | Absence of dissolved $0_2$ and $NO_3^-$<br>Anaerobic zone<br>Fermentation | FERMENTATIVE PATHWAY<br>→ Volatile Acids | ENHANCED REDUCTION<br>→ $NH_4^+$ |
| - 500 mV |  | FERMENTATIVE PATHWAY<br>→ Methane |  |

variation, the air flow rate could be adjusted to increase the oxygen supplied to attain an optimum ORP level (~200 mV) to optimize nitrification or reduce the air flow rate in case of low effluent ammonium. Similarly, a low ORP limit can be used to regulate a reductive environment for denitrification.

Automated systems designed for finer analysis of ORP by redox sensors include the logic controller SUPEROXYDOSE and the REGUL'N® (Charpentier *et al.*, 1998). The REGUL'N® includes ORP set points overruled by on-line effluent nitrogen sensors, either ammonium or nitrate, according to the applicable effluent regulation.

# Chapter 9
# Case Studies

# Chapter 9
# Case Studies

This chapter provides examples of Canadian and international case studies of ammonia removal applications at MWWTPs. A comprehensive description is provided of two relevant case studies relating to the Little River MWWTP in the City of Windsor, Ontario (Section 9.1.1), and the Saint-Nicolas MWWTP in the vicinity of Quebec City (Section 9.1.2). Brief summaries of relevant information from a number of other MWWTPs covering various process types and treatment scenarios involving ammonia removal are subsequently provided in Section 9.1.3 (i.e., tabulated summaries in Tables 9.3 and 9.4). The summary information was based on investigations and references to the literature.

## 9.1 Investigated Case Studies

### 9.1.1 Comprehensive Case Study No. 1: Little River MWWTP Process Evaluation and Optimization

#### 9.1.1.1 Background

A comprehensive investigation involving the use of process auditing techniques was conducted at the City of Windsor's Little River MWWTP, which faced a possible derating to achieve nitrification. This case study is an example of the significant benefits of process audits for the determination of actual existing plant capacity and treatment capabilities resulting in plant rerating (Newbigging *et al.*, 1994; CG&S and Hydromantis, Inc., 1996). It demonstrated that, although loading rates applied to the existing treatment plant during the process evaluations exceeded guideline levels for nitrification, a satisfactory level of treatment could be achieved through optimization and improved process control (WERF, 1999).

#### 9.1.1.2 Facility Description

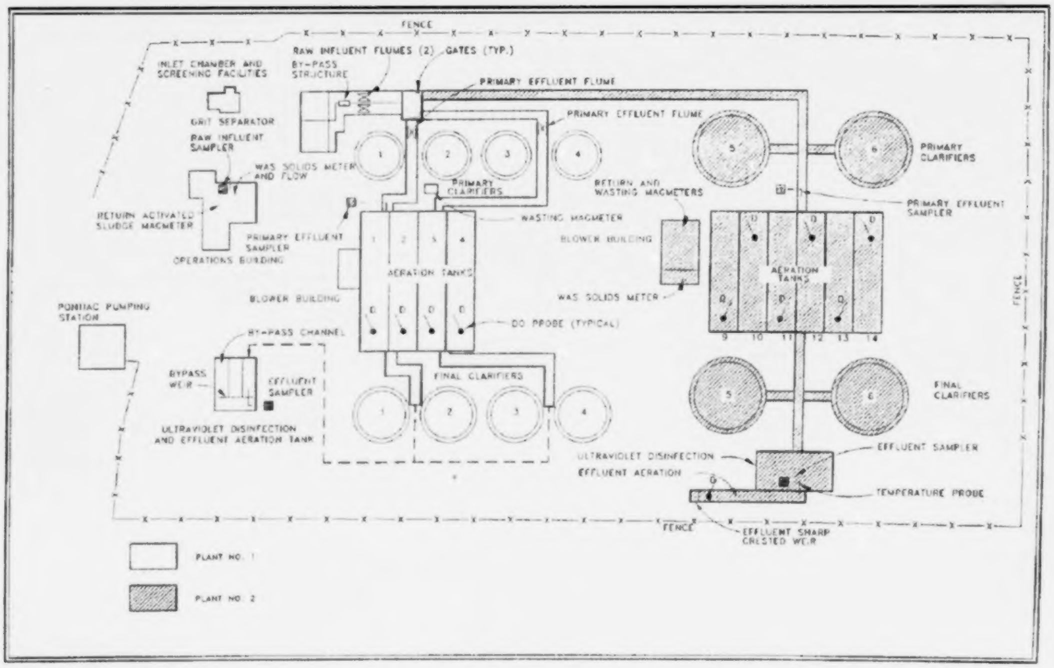
At the time of the investigation, the Little River MWWTP provided secondary treatment in two separate conventional activated sludge plants (i.e., Plants No. 1 and 2) with an overall nominal design capacity of 63.7 MLD (14 MIGD). The facility treated an average flow rate of 38.2 MLD (i.e., 8.4 MIGD) during this period of investigation between 1989 and 1993. A schematic layout of the Little River MWWTP is shown in Figure 9.1.

The Little River MWWTP provides combined raw sewage pumping, screening, grit removal and aluminum addition for phosphorus removal prior to splitting the flow between the two plants. The two parallel plants include enhanced primary treatment, conventional activated sludge secondary treatment and ultraviolet disinfection.

Plant No. 1 (36.4 MLD nominal rating and a peaking factor of 2) consisted of four parallel activated sludge aeration tanks with a total volume of 4900 $m^3$ and a corresponding design average HRT of 3.2 hours. The aeration stage employed mechanical aeration in one of the aeration tanks and fine-pore diffused aeration in the other three tanks. The secondary clarification stage consisted of four circular sedimentation tanks with 1870 $m^2$ total surface area and a design surface overflow rate of 38.9 $m^3/m^2$ per day at peak flow. Plant No. 1 was not originally intended to nitrify.

Plant No. 2 (27.3 MLD nominal rating and a peaking factor of 2) was designed to nitrify and includes six parallel aeration tanks having a total volume of 7634 $m^3$ and a design average HRT of 6.0 hours. The aeration tanks are equipped with a fine-pore aeration system and include provision for an up-front anoxic zone and step feed operation. The secondary clarification stage

**Figure 9.1 Schematic Layout of the Little River MWWTP**


FENCE
RAW INFLUENT FLUMES (2)
GATES (TYP.)
INLET CHAMBER AND SCREENING FACILITIES
BY-PASS STRUCTURE
PRIMARY EFFLUENT FLUME
GRIT SEPARATOR
RAW INFLUENT SAMPLER
PRIMARY EFFLUENT FLUME
1
2
3
4
5
6
PRIMARY CLARIFIERS
WAS SOLIDS METER AND FLOW
PRIMARY CLARIFIERS
RETURN AND WASTING MAGMETERS
PRIMARY EFFLUENT SAMPLER
RETURN ACTIVATED SLUDGE MAGMETER
WASTING MAGMETER
PRIMARY EFFLUENT SAMPLER
OPERATIONS BUILDING
BLOWER BUILDING
AERATION TANKS
AERATION TANKS
BLOWER BUILDING
WAS SOLIDS METER
FENCE
PONTIAC PUMPING STATION
D
DO PROBE (TYPICAL)
9
10
11
12
13
14
BY-PASS CHANNEL
BYPASS WEIR
EFFLUENT SAMPLER
FINAL CLARIFIERS
FINAL CLARIFIERS
ULTRAVIOLET DISINFECTION AND EFFLUENT AERATION TANK
ULTRAVIOLET DISINFECTION
EFFLUENT SAMPLER
EFFLUENT AERATION
TEMPERATURE PROBE
FENCE
EFFLUENT SHARP CRESTED WEIR
PLANT NO. 1
PLANT NO. 2

comprises two circular sedimentation tanks with a total surface area of 2200 $m^2$ and a design surface overflow rate of 24.8 $m^3/m^2$ per day at peak flow. A 640-kW blower system used to supply air to the two plants had a rated oxygenation capacity of 21 tonnes of oxygen per day (with 0 mg of residual DO per litre).

### 9.1.1.3 Process Audit Findings

Prior to the construction of Plant No. 2 in 1988, a conditional certificate of approval was issued, which required the overall facility to achieve organic carbon removal as well as nitrification, including the ability to handle peak flows. This raised questions concerning the actual capacity of the existing plant (i.e., Plant No. 1) and the new plant (i.e., Plant No. 2). A down-rating of the overall facility to 45.5 MLD was a possibility in order to meet the enhanced treatment requirement including nitrification.

Importantly, the process audit and optimization work revealed that with accurate and consistent SRT control, together with some physical upgrades to Plant No. 1 (i.e., replacing the mechanical aerators in the one tank with fine-pore diffusers, installation of baffles in the secondary clarifiers and implementation of an RAS chlorination system for filamentous control), the facility could achieve year-round nitrification at hydraulic loading rates greater than the original facility rating. The evaluation findings revealed that SRT must be maintained at a set point of higher than 7 days to achieve an effluent total ammonia-nitrogen concentration of less than 2 mg/L throughout the year, with highest SRT levels required in the winter to offset the impact of colder wastewater temperature. A sludge wasting model was developed and implemented to estimate the required wasting rate based on the last 7 days of process data for smooth control of SRT.

As part of the process audit, the performance of each of the two plants was separately evaluated at elevated hydraulic, organic and nitrogen loading rates. Stress testing involved an increase in loading rates to the two plants by taking some of the existing tankage out of service. Some of the other auditing techniques that were used in the determination of the actual capacity of each of the two plants included:

- historical process operational and performance data review;
- off-line sampling and analysis, including an extensive quality assurance/quality control (QA/QC) program;
- on-line monitoring;
- aeration system oxygen transfer testing;
- flow metering evaluation;
- microscopic analysis of microorganisms; and
- implementation and assessment of a sludge wasting control strategy.

#### 9.1.1.3.1 Evaluation of Plant No. 1

Stress testing of Plant No. 1 was conducted over an 8-month period to evaluate the plant's ability to achieve the effluent discharge criteria during winter, spring and summer/fall. These three periods were selected to cover the range of operating conditions experienced at the facility, including cold wintertime weather, which may impact nitrification, high influent flow rates associated with wet weather during spring, and increased influent organic loadings associated with peak industrial effluent loads from a local cannery during late summer and early autumn. During stress testing, between 60 and 70% of the total wastewater flow was directed into one-half of Plant No. 1. The performance of the stressed plant was determined using on-line instrumentation data and results of an off-line sampling and chemical analysis program, which included the measurement of conventional parameter concentrations in the influent, primary effluent, treated effluent from the stressed plant and the combined plant effluent.

The stress testing and oxygen transfer testing demonstrated that there was no reason to down-rate the average daily flow capacity of Plant No. 1, since it was capable of achieving all effluent requirements, including nitrification, at the original rating of 36.4 MLD. The results of the stress testing represented a worst-case scenario with respect to contaminant loadings, since the testing typically involved average influent concentrations at peak levels of applied hydraulic loading, while actual influent concentrations (especially TKN and total ammonia-nitrogen) during storm events would tend to be lower than on an average day.

The stress testing revealed that the plant's ability to maintain virtually complete nitrification (i.e., <2 mg of total ammonia-nitrogen per litre) was affected at a peak-day hydraulic loading rate during the winter, but that the plant effluent would still meet effluent discharge requirements (i.e., discharge limits included a monthly average and a daily non-compliance concentration of 6.0 and 8.0 mg total ammonia-nitrogen/L, respectively). The overall average and maximum monthly average total ammonia-nitrogen concentrations in the treated effluent during the investigation were 2.2 and 5.8 mg/L, respectively.

#### 9.1.1.3.2 Evaluation of Plant No. 2

A similar approach to stress testing was used in the subsequent evaluation of Plant No. 2.

Approximately one-third of the aeration capacity and one-half of the primary and secondary clarification capacity in Plant No. 2 were in operation for a 13-month period of intensive evaluation. The aeration tank HRT, F:M ratio and SRT averaged 3.5 hours, 0.22 kg BOD$_5$/kg MLVSS per day and 8 days, respectively, over a 6-month period, and the surface overflow rate and solids loading rate to the secondary clarification stage exceeded the Ontario Ministry of the Environment guidelines (OMOE, 1984) on a number of occasions. The process audit included on-line monitoring and off-line sampling and characterization and the assessment of the oxygenation capacity of the aeration system. It involved a review of the step feed operations for Plant No. 2 and an assessment of the capacity and effectiveness of the ultraviolet disinfection system.

Seven distinct periods of operation were identified, including the winter, spring and summer/fall seasonal periods, different hydraulic loading rate conditions and process upset situations (e.g., one period of excessive sludge wasting and one period involving excessive organic loading from the cannery). Raw sewage and primary effluent TKN averaged 26 and 22 mg/L, respectively. The corresponding overall average and maximum monthly average total ammonia-nitrogen concentrations in the treated effluent were 0.9 and 3.1 mg/L, respectively. Based on the stress testing results, it was recommended that the rating for Plant No. 2 be increased to 36.4 MLD.

### 9.1.1.4 Estimated Cost Savings

Using the CapdetWorks™ software program by Hydromantis, Inc., and based on the available information from the process evaluations, the equivalent construction cost savings associated with the increased available treatment capacity determined for the Little River MWWTP are estimated to be about $4.6 million. This estimated total construction cost savings are actualized to present-day Canadian dollars and include all construction, civil works, site preparation, site electrical, yard piping, I&C, etc. associated with a physical expansion of the two existing plants that would have been required to match Ontario Ministry of the Environment design guidelines for nitrification (OMOE, 1984) for a total capacity of 72.8 MLD. Physical dimensions of the two existing activated sludge plants as well as historical influent characteristics were used as input data to the software program.

The estimated $4.6 million construction cost savings include $3.5 million associated with an expansion of both the aeration stage and the secondary clarification stage that would have been required for Plant No. 1 to meet the Ontario Ministry of the Environment design guidelines. The remaining $1.1 million cost savings are associated with an aeration stage expansion at Plant No. 2 that would have been required to provide a 6.0-hour HRT at the proposed rerated flow rate of 36.4 MLD. No expansion of the secondary clarification stage would have been required to meet the Ontario Ministry of the Environment guidelines for this increased flow rate.

### 9.1.1.5 Summary

The comprehensive process audit demonstrated that the Little River MWWTP did not need to be derated to a 45.5-MLD capacity for nitrification. Actually, nitrification could be achieved at relatively low aeration tank HRTs of between 2.6 and 3.5 hours at this plant. The facility was found to possess a demonstrated average hydraulic capacity of 72.8 MLD at the prevailing influent characteristics and operating conditions and effluent non-compliance requirements at the time of the study.

During the investigations, although applied process loadings to the aeration and secondary clarification stages of the two plants tended to exceed values recommended by the Ontario Ministry of the Environment's wastewater treatment plant design guidelines for nitrifying activated sludge processes (OMOE, 1984; see Appendix A), satisfactory performance including nitrification was achieved. The stress testing identified upgrading options for the overall facility to provide increased operational flexibility and to further increase total plant capacity (CG&S and Hydromantis, Inc., 1996). For example, the Little River MWWTP has provided the operators with process control training and additional on-line instrumentation and control equipment to assist in optimizing the performance of the treatment processes.

The demonstrated increase in firm capacity of the treatment facility provides the city with ample capacity for future growth and more than justified the effort and resources expended on the process evaluation and optimization program. By expanding the treatment system rating, the city realized capital savings on the order of several million dollars of avoided construction.

## 9.1.2 Comprehensive Case Study No. 2: Saint-Nicolas (Québec) MWWTP

### 9.1.2.1 Background

The Saint-Nicolas MWWTP is located on the south shore of the St. Lawrence River in the vicinity of Quebec City. The plant treats the wastewaters from three municipalities — Saint-Nicolas, Charny and Saint-Rédempteur. An industrial portion is included in the municipal wastewaters and represents 10–15% of the total influent load to the treatment plant. The industries discharging their process waters in the municipal sewers include two food processing industries and an aluminum processing plant with physical/chemical pretreatment facilities on the plant site.

SBRs are operated at the Saint-Nicolas MWWTP and offer a relatively high degree of process flexibility and control over process conditions and cycle times. The SBR biological treatment and the sludge process installations offer the potential for complete biological nutrient removal. The treatment plant achieves $BOD_5$ and suspended solids removal below the discharge requirement levels and sustains nitrification activity. Phosphorus removal is currently achieved by normal biomass phosphorus uptake. However, EBPR could be implemented when required, as the treatment plant has the equipment facilities and process control for this advanced wastewater treatment.

The municipal wastewater characteristics and the treatment requirements at the Saint-Nicolas treatment plant are summarized in Table 9.1.

### 9.1.2.2 Treatment Process Options

Two main process technologies were evaluated at the preliminary stages of the project. Biofiltration and SBRs were identified as two technologies that offer biological secondary treatment and nitrification, if required. Both technologies are modular and operate without secondary clarification and RAS steps (Section 5.1.3.1, Section 5.1.4 and Appendix A).

Initially, high flow rates measured during springtime were associated with high water table conditions. The water table conditions were redefined during the preliminary stages, and the high flow rates previously associated with high water table conditions were then considered as spring thaw flow rates occurring over a shorter period of time, since they were measured in April. The flow rates measured in early May were then related to high water table conditions instead of average water table. The water table conditions evaluation and the specific period for the flow rate

## Table 9.1 Saint-Nicolas MWWTP Influent Characteristics and Discharge Requirements

| Parameters | Influent Final Design Criteria (Design Reference Year: 2002) | Influent Actual Average Parameter Measurements (2000) | Effluent Discharge Requirements to the Chaudière River |
|---|---|---|---|
| Population | 35 950 | 34 150 | |
| Flow rate ($m^3/d$) | 19 480 | 13 400[1] | |
| $BOD_5$ (mg/L) | 114 | 108 | 30 (monthly)<br>20 (yearly) |
| Suspended solids (mg/L) | 125 | 183 | 30 (monthly)<br>20 (yearly) |
| TKN (mg/L) | 21 | 20 | —[2] |
| $NH_4^+$-N (mg/L) | 9 | 14 | —[2] |
| Total phosphorus (mg/L) | 3 | 4 | 1.0 (monthly, May 15 to November 15)<br>0.8 (from May 15 toNovember 15) |
| Fecal coliforms (organisms/100 mL) | | | 600 (June 1 to September 30)<br>1500 (May and October) |

[1] The value is the average treated flow rate at the MWWTP.
[2] No current discharge requirements.

measurements have an effect upon the biological process selection.

Biofiltration is an attached-growth process that offers the possibility of simultaneously achieving biological treatment and suspended solids filtration as the wastewaters pass through the depth of the medium. Specific considerations related to biofiltration application included the following:

- The attached-growth system combined with a filtration process within the same reactor provides a more stable and reliable treatment performance, with improved resistance to process upsets.
- Biofiltration allows high hydraulic condition periods to be sustained with diluted waters.
- Biomass is significantly less susceptible to microorganism wash-out during hydraulic peaks and low $BOD_5$ and nutrient periods, compared with suspended-growth biomass, due to the biofilm physical attachment to the media.

SBRs are a variation of the suspended-growth activated sludge process. SBRs are characterized by discontinuous and variable-volume batch feed of parallel reactors. Each reactor operates through a treatment cycle that involves initial batch feed, aeration/mix-only periods for biological treatment, settling of the suspended-growth biomass and decanting of the supernatant treated effluent. The mixed liquor clarification stage occurs in the aerated basins by interrupting the aeration and mixing. Specific considerations related to SBR application included the following:

- SBRs provide enhanced process flexibility and higher degree of operational process control;

thus, the system is inherently more adaptable to a variety of process configurations.

- Prolonged high flow rates and diluted water conditions may cause operational difficulties, and potential biomass wash-out can be a serious concern.
- Control over process conditions (aerobic/anoxic/anaerobic) and cycle times of the various phases for microorganism selection may favour advanced tertiary treatment efficiency, including nitrification, denitrification and EBPR.
- Aeration phase length control may provide, more specifically, a longer aerated period to favour nitrification process completion.
- Disinfection may be more difficult to implement with SBR than biofiltration due to discontinuous decantation of the reactors and may require treated effluent buffering.

In terms of hydraulic conditions, the high flow rates initially associated with high water table conditions indicated that diluted waters could occur over an extended period of time and thus favour the biofiltration technology. The revision of the water table conditions reallocated the high flow rates previously associated with high water table conditions to spring thaw flow rates. Thus, the spring thaw high flow rates and diluted waters are occurring over a shorter period of time and are less harmful to suspended-growth SBR operation.

SBR process potential for EBPR offers several advantages compared with chemical phosphorus removal technology that would be required with the biofiltration process. The major EBPR benefits include elimination/reduction of chemical addition for phosphorus removal, excess sludge generation reduction, easier sludge processing and potential for excess sludge land application (Section 5.3.2.1). Improved nitrification rates in the aerobic phase may also be obtained by removing much of the soluble organic substrate in the anaerobic phase.

The technologies were evaluated to establish order-of-magnitude cost comparisons between both options. The costing evaluation indicated that SBR construction and operation costs were, respectively, 19% and 8% lower than those for biofiltration. The actualized combined construction and operation costs for SBRs were 15% lower than those for biofiltration.

Considering the revised hydraulic conditions, the process flexibility and control associated with the SBR technology and the lower cost for SBR construction and operation, the SBRs were recommended for the Saint-Nicolas MWWTP application.

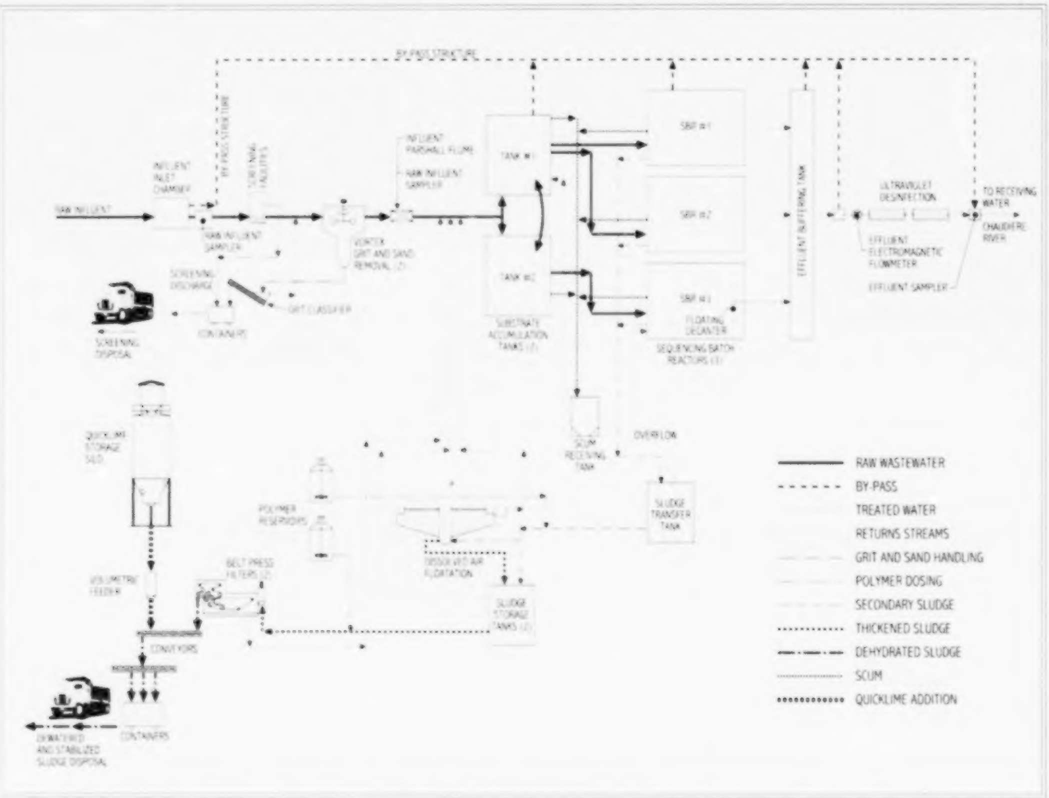
### 9.1.2.3 Facility Description

The Saint-Nicolas MWWTP schematic layout is presented in Figure 9.2, and the major process units are briefly summarized in this section. The wastewater treatment processes include pretreatment, biological secondary and tertiary treatment, and disinfection. The sludge treatment units include excess sludge thickening, dewatering and stabilization.

The raw wastewaters first enter an influent inlet chamber that provides some control over the maximum wastewater flows prior to the pretreatment units. Two vertical screens followed by two vortex grit and sand removal units achieve the required pretreatment. The wastewaters then pass through a Parshall flume for flow rate measurements.

Two substrate accumulation tanks receive the pretreated wastewaters. These basins serve two objectives. The first one is to equalize the hydraulic and organic peak loads from the municipal and industrial wastewaters and from

**Figure 9.2 Schematic Layout of the Saint-Nicolas MWWTP**


BY-PASS STRUCTURE
BY-PASS STRUCTURE
RAW INFLUENT
INFLUENT INLET CHAMBER
SCREENING FACILITIES
RAW INFLUENT SAMPLER
VORTEX GRIT AND SAND REMOVAL (2)
INFLUENT PARSHALL FLUME
RAW INFLUENT SAMPLER
SCREENING DISCHARGE
GRIT CLASSIFIER
CONTAINERS
SCREENING DISPOSAL
TANK #1
TANK #2
SUBSTRATE ACCUMULATION TANKS (2)
SBR #1
SBR #2
SBR #3
FLOATING DECANTER
SEQUENCING BATCH REACTORS (3)
EFFLUENT BUFFERING TANK
ULTRAVIOLET DESINFECTION
EFFLUENT ELECTROMAGNETIC FLOWMETER
EFFLUENT SAMPLER
TO RECEIVING WATER
CHAUDIÈRE RIVER
SCUM RECEIVING TANK
OVERFLOW
QUICKLIME STORAGE SILO
POLYMER RESERVOIRS
DISSOLVED AIR FLOATATION
SLUDGE TRANSFER TANK
SLUDGE STORAGE TANKS (2)
BELT PRESS FILTERS (2)
VOLUMETRIC FEEDER
CONVEYORS
CONTAINERS
DEWATERED AND STABILIZED SLUDGE DISPOSAL
RAW WASTEWATER
BY-PASS
TREATED WATER
RETURNS STREAMS
GRIT AND SAND HANDLING
POLYMER DOSING
SECONDARY SLUDGE
THICKENED SLUDGE
DEHYDRATED SLUDGE
SCUM
QUICKLIME ADDITION

the sludge process return sidestreams. The second objective is to provide the potential for some level of prefermentation within the substrate accumulation tanks. This biological activity will favour the presence and availability of readily biodegradable COD substrate during the first anaerobic phase of the SBR cycle in the case of EBPR process implementation.

Air jet headers with dedicated positive displacement blowers and submersible centrifugal recirculation pumps are installed in the substrate accumulation tanks. This type of arrangement permits mixing with or without aeration of the wastewater content. In the case of EBPR, mixing periods without aeration would favour prefermentation activities. The substrate accumulation tanks are currently mainly operated for equalization purposes, as EBPR is not yet implemented at the treatment plant.

Three SBRs receive the equalized wastewaters. Air jet headers with dedicated positive displacement and variable-speed drive blowers, submersible recirculation pumps and floating decanters are installed in the SBRs. Jet aeration permits independent control of mixing and aeration of the mixed liquor. Each SBR is operated with an average of four cycles per day. The sequential operating phases of an SBR cycle are presented in Table 9.2 and permit the oxidation of organic pollutants ($BOD_5$ and total ammonia-nitrogen) but not EBPR.

Batch feed of the SBRs is achieved within a short period of time and is followed by a mixed anoxic phase. Thus, a high initial F:M ratio is experienced and an anoxic selective effect to prevent the growth of filamentous organisms is achieved. The batch feed may also favour higher biological reaction rates. The short mixed-only phase provides some predenitrification activity (Section 5.3.1.2).

The react phase provides the aerobic environment required for $BOD_5$ oxidation and nitrification activity. The DO level in the mixed liquor is maintained between 1 and 3 mg/L. Variable-speed drive blowers are automatically adjusted according to continuous DO concentration measurements. Thus, at 1 mg DO/L, the blowers operate at 100% speed; from 1 to 3 mg DO/L, the blowers' speed is gradually reduced; at 3 mg DO/L, the blowers are stopped and the react phase continues with mixing provided by the recirculation pumps. According to the nitrification efficiency at the MWWTP, this aeration control provides sufficient DO to sustain nitrification.

The SBRs are operated with an average F:M ratio of 0.07 (kg $BOD_5$/kg MLVSS per day) and an average HRT of 19 hours. The SRT varies from 10 to 20 days with an average MLSS concentration of 3300 mg/L to sustain the development of a nitrifying bacteria population.

Following a settling phase of the mixed liquor, the treated supernatant is discharged during the decant phase to an effluent buffering tank. This basin permits the water to be gradually passed through the ultraviolet disinfection units prior to final discharge.

The excess activated sludge generated by the biological process is wasted from the SBRs at every cycle and collected first in a sludge transfer tank. The excess sludge HRT in the transfer tank varies from 10 to 14 hours, and the excess sludge is continuously mixed and aerated. The DO level varies from 2 to 5 mg/L. In the case of EBPR, the excess sludge would have to be aerated through all the sludge treatment process to avoid anaerobic phosphorus release, and the excess sludge HRT would also need to be maintained as low as possible to avoid aerobic digestion.

The sludge is then transferred to a dissolved air flotation unit for thickening. The thickened sludge is stored in sludge storage tanks with an HRT of 24–46 hours and is continuously mixed and aerated. The average DO level is 0.5 mg/L. The thickened sludge is then dewatered with belt press filters and stabilized with quicklime prior to final disposal.

#### 9.1.2.4 Process Optimization

The treatment plant operators are in the process of optimizing the SRT and the MLSS concentration adjustments for operation of the SBRs under critical conditions to maintain optimal nitrification all year long according to seasonal wastewater temperature.

#### 9.1.2.5 Estimated Cost

The complete Saint-Nicolas MWWTP actual construction cost, including corrective work and engineering, was $18 million.

### *9.1.3 Case Study Summaries*

Tables 9.3 and 9.4 provide a summary of various other pertinent MWWTP applications that either have been designed for consistent nitrification or have been upgraded to achieve year-round nitrification. Specific details are provided in the remarks column of the tabulated summaries

**Table 9.2 Duration of Cycle Phases at the Saint-Nicolas MWWTP**

| Phases | Operations | Length (minutes) |
|---|---|---|
| Static fill | Rapid influent batch feed to idle basin (without mixing and aeration) | 13–15 |
| Mixed only | Influent batch feed stopped and SBR mixing without aeration (anoxic conditions) | 15 |
| React | SBR mixing with aeration (aerobic conditions) | 80-205 [1] |
| Presettling | Jet aeration header backwash (once every 4 days) | 7 |
| Settling | Mixing and aeration stopped for mixed liquor clarification | 60–65 |
| Decant | Clarified supernatant discharge to the effluent buffering tank | 50–120 [2] |
| Excess WAS | Initiated 15 minutes following the beginning of the decant phase | Varies with settled MLSS concentration and SRT requirements |
| Idle | SBR on standby and awaiting refilling and a repeat of phases | Variable [3] |

[1] React phase length varies according to substrate accumulation tank water level fluctuations (influent flow rate variations).
[2] Decant phase length varies according to clarified supernatant volume to discharge and effluent buffering tank water level.
[3] Idle phase length varies according to substrate accumulation tank water level fluctuations (influent flow rate variations).

of investigated and reported case studies, respectively.

In addition to the full-scale applications described, a summary is provided of findings of the extensive pilot plant investigation at the West Windsor MWWTP to assess various nitrification processes, as was described in Section 6.3.2.2. The nitrification systems studied included BAF, trickling filter/solids contact, RBC and conventional and modified activated sludge processes. This work is reported in City of Windsor (1992, 1996).

## 9.2 Reported Case Studies

Various additional reported case studies specific to nitrification and biological nutrient removal systems can be found in USEPA (1993), Daigger and Buttz (1998) and WEF (1998).

A report by WTC and VPISU (1994) included a number of case studies summarizing planned or installed secondary treatment plant upgrades using conventional and innovative retrofit technologies to achieve year-round nitrification of municipal wastewaters.

A report by Hydromantis, Inc. and SBR Technologies Inc. (1998) discusses the ability of SBR plants to consistently achieve a monthly average total ammonia-nitrogen discharge concentration of <3 mg/L on a year-round basis. It also provides cost information and other summary information on full-scale applications, including five SBR-based MWWTPs in Canada.

## Table 9.3 Summary of Investigated Case Studies of Ammonia Removal at Selected MWWTPs

| Wastewater Treatment Plant and Location | Treatment Technologies | Design Capabilities | Design Average Flow Rate | Remarks | Contact Person |
|---|---|---|---|---|---|
| Cardinal, Ontario | Sequencing batch reactor (SBR) | Carbon removal — Nitrification | 2500 $m^3/d$ | Raw influent characterization (average year 2000):<br>- average flow rate: 1695 $m^3/d$<br>- $BOD_5$: 31 mg/L (caused by infiltration)<br>- suspended solids: 64 mg/L<br>- TKN: 8 mg/L<br>- total phosphorus: 2.7-3.0 mg/L<br><br>Effluent requirements:<br>- $BOD_5$: 25 mg/L compliance, 15 mg/L objective<br>- suspended solids: 25 mg/L compliance, 15 mg/L objective<br>- total phosphorus: 1 mg/L compliance<br>- ammonium: no compliance for ammonia<br><br>Effluent characterization (average year 2000):<br>- $BOD_5$: 1.5 mg/L<br>- suspended solids: 1.5 mg/L<br>- TKN: 1.2 mg/L (year-round ammonia removal)<br>- total phosphorus: 0.8 mg/L (phosphorus removal by normal biological phosphorus uptake)<br><br>Configuration:<br>- primary treatment: screening<br>- sand and grit removal<br>- secondary treatment: sequencing batch reactors (two reactors)<br>- disinfection: ultraviolet<br>- sludge treatment: autothermal thermophilic aerobic digesters<br><br>Costs:<br>- complete treatment plant construction: $6.5 million<br>- O&M: $210,000 per year (includes material, electricity, heat, salaries) | Mr. Jim Grant<br>Chief Operator<br>Tel.: (613) 657-3765 |
| Canmore, Alberta | Upflow biofilter (BAF), two stages | Carbon removal — Nitrification | 9700 $m^3/d$ | Raw influent characterization (average year 2000):<br>- average flow rate: 5260 $m^3/d$<br>- $BOD_5$: 297 mg/L<br>- suspended solids: 302 mg/L<br>- total ammonia-nitrogen: 16.6 mg/L<br>- total phosphorus: 7.1 mg/L<br>- temperature: winter: 11.9°C; summer: 14.7°C<br><br>Effluent requirements:<br>- $cBOD_5$: 20 mg/L compliance (monthly)<br>- suspended solids: 20 mg/L compliance (monthly)<br>- total phosphorus: 1.0 mg/L compliance (monthly)<br>- ammonium: October 1 to June 30: 10 mg/L compliance (monthly); July 1 to September 30: 5.0 mg/L compliance (monthly) | Mrs. Tami Wetmore<br>Service Manager<br>EPCOR<br>Canmore Operations<br>102 Bow Valley Trail<br>Canmore, Alberta<br>T1W 3B7<br>Tel.: 403-609-4777<br>Fax: 403-609-4790<br>E-mail:<br>twetmore@epcor.ca |

## Table 9.3 Summary of Investigated Case Studies of Ammonia Removal at Selected MWWTPs *Contd.*

| Wastewater Treatment Plant and Location | Treatment Technologies | Design Capabilities | Design Average Flow Rate | Remarks | Contact Person |
|---|---|---|---|---|---|
| | | | | Effluent characterization (average year 2000):<br>- cBOD$_5$: 5.9 mg/L<br>- suspended solids: 6.0 mg/L<br>- total ammonia-nitrogen: October 1 to June 30: 2.4 mg/L; July 1 to September 30: 1.2 mg/L<br>- TKN: October 1 to June 30: 5.5 mg/L; July 1 to September 30: 5.6 mg/L<br>- total phosphorus: 0.7 mg/L (phosphorus removal with alum coagulation-precipitation)<br><br>Configuration:<br>- Primary treatment:<br>  - travelling screens<br>  - grit removal<br>  - alum injection and precipitation<br>  - primary clarifier<br>- Secondary treatment:<br>  - four parallel trains with two Biofor (BAF) per train<br>  - first stage includes four Biofor C cells for carbonaceous removal of 40 m$^2$ each with 2.5 m depth of biolite N media of 2.7-mm effective size<br>  - second stage includes four Biofor N cells for nitrification with the same characteristics<br>  - all cells can be switched to parallel operation under peak flow events<br>- Sludge treatment:<br>  - aerobic digestion<br>  - belt filter dewatering<br><br>Costs:<br>- complete treatment plant construction: $16 million<br>- O&M: not available. | |
| Guelph, Ontario | Activated sludge, upgraded with tertiary RBCs | Carbon removal — Nitrification | 61 000 m$^3$/d | Raw influent characterization (average 1999):<br>- flow rate: 50 450 m$^3$/d (about 45% industrial and 55% domestic)<br>- cBOD$_5$: 235 mg/L<br>- suspended solids: 280 mg/L<br>- TKN: 31.8 mg/L<br><br>Effluent requirements (maximum discharge concentrations):<br>- cBOD$_5$: 10.8 mg/L<br>- suspended solids: 10.8 mg/L<br>- total ammonia-nitrogen: 5.4 mg/L | Mr. S. Wayne Key<br>Superintendent of Wastewater Services<br>Works Department<br>The City of Guelph<br>Guelph, Ontario<br>Tel.: 519-837-5629 |

### Table 9.3 Summary of Investigated Case Studies of Ammonia Removal at Selected MWWTPs *Contd.*

| Wastewater Treatment Plant and Location | Treatment Technologies | Design Capabilities | Design Average Flow Rate | Remarks | Contact Person |
|---|---|---|---|---|---|
| | | | | Treatment effluent characteristics (monthly averages in 1999):<br>- cBOD$_5$: 2–4 mg/L<br>- suspended solids: 2-5 mg/L<br>- total ammonia-nitrogen: 0.7–4.9 mg/L<br>Configuration in 2000:<br>- preliminary treatment by screening and degritting<br>- primary treatment by sedimentation clarification<br>- secondary treatment by conventional activated sludge (i.e., three parallel plants with 13 400 m$^3$/d total aeration basin volume, achieving partial nitrification during the summertime only), including chemical phosphorus control<br>- tertiary treatment by nitrifying RBCs (i.e., four parallel tanks with eight discs per tank for a total media surface of 440 000 m$^2$; air-driven by two 75 horsepower blowers) and sand filtration<br>- disinfection by chlorination/dechlorination<br>- sludge processing by two-stage mesophilic anaerobic digestion, dewatering and in-vessel composting<br>- recirculation of filtrate, supernatant and compost off-gas<br>- energy recovery by cogeneration<br>Costs:<br>- total construction cost for the add-on nitrifying RBC process stage was approximately $5 million Canadian in 1978; annual electricity cost to run the RBC process is about $96,000 | |
| West Windsor, Ontario (pilot plant studies) | BAF, trickling filter/solids contact, RBC, activated sludge | Carbon removal — Nitrification | Various (pilot scale) | Raw influent characterization:<br>- effluent supply from chemically assisted primary treatment plant<br>Primary effluent characterization:<br>- BOD$_5$: 43 mg/L<br>- suspended solids: 23 mg/L<br>- total ammonia-nitrogen: 9.5 mg/L<br>Effluent objectives (maximum monthly averages):<br>- BOD$_5$: 15 mg/L<br>- suspended solids: 15 mg/L<br>- total ammonia-nitrogen: 3 mg/L | Mr. Michael Newbigging<br>Senior Project Manager and Principal (was the pilot plant project manager)<br>Hydromantis, Inc.<br>210 Sheldon Drive<br>Cambridge, Ontario<br>N1T 1A8 |

## Table 9.3 Summary of Investigated Case Studies of Ammonia Removal at Selected MWWTPs *Contd.*

| Wastewater Treatment Plant and Location | Treatment Technologies | Design Capabilities | Design Average Flow Rate | Remarks | Contact Person |
|---|---|---|---|---|---|
| | | | | Treatment performance (degree of compliance with the effluent objectives by the respective trickling filter/solids contact, RBC, BAF and activated sludge processes):<br>- 50%, 100%, 100% and 83% for BOD<br>- 66%, 100%, 100% and 83% for suspended solids<br>- 100%, 90%, 80% and 100% for total ammonia-nitrogen<br>All of the processes were able to handle peak day and diurnal flow variations.<br>Based on a comparative evaluation with respect to various categories, the BAF process scored highest for the proposed West Windsor MWWTP upgrade application, followed in order by the RBC, activated sludge and trickling filter/solids contact processes. The categories included effluent performance, hydraulic performance, solids handling, operational aspects and economics.<br>References: City of Windsor (1992, 1996) | |

## Table 9.4 Summary of Reported Case Studies of Ammonia Removal at Selected MWWTPs

| Wastewater Treatment Plant and Location | Treatment Technologies | Design Capabilities | Design Average Flow Rate | Remarks | Contact Person |
|---|---|---|---|---|---|
| Annapolis Water Reclamation Facility, Maryland, USA | Hybrid activated sludge; upgrade with Ringlace | Carbon removal — Nitrification — Denitrification | 38 000 m$^3$/d | Raw influent characterization (average: 10/96 to 04/98):<br>- average raw influent flow rate: 32 000 m$^3$/d<br>Primary effluent characterization (average: 10/96 to 04/98):<br>- average primary effluent flow rate: 34 400 m$^3$/d<br>- BOD$_5$: 111 mg/L<br>- suspended solids: 79 mg/L<br>- ammonium:16.5 mg/L<br>- TKN: 19 mg/L<br>Effluent requirements:<br>- total nitrogen: 8 mg/L objective<br>Temperature in activated sludge basins:<br>- 15.0°C in winter, 24.5°C in summer<br>Effluent characterization (average: 10/96 to 04/98):<br>- BOD$_5$: 17 mg/L<br>- suspended solids: 11 mg/L<br>- ammonium: 1.4 mg/L (year-round ammonia removal)<br>- total nitrogen: 11.0 mg/L<br>Configuration:<br>- Primary treatment:<br>- fine screening<br>- two grit removal<br>- two primary clarifiers<br>- Secondary treatment:<br>- one U-shape activated sludge basin with:<br>- step feed<br>- two sections: first anoxic and second aerobic in series<br>- mixed liquor nitrate recirculation at anoxic front-end<br>- 109 000 m of Ringlace media in aerobic and anoxic sections in two blocks of racks<br>- RAS<br>- two secondary clarifiers<br>- Disinfection: chlorination and dechlorination<br>Costs:<br>- upgrade costs: $14 million,[1] including 244 000 m of media (Ringlace), two U-shape basins upgraded in four parallel trains with aerobic and anoxic zones, two additional secondary clarifiers, instrumentation upgrade, one gravity thickener, and new RAS and WAS pumps | Mr. Greg Farren<br>Facility Manager<br>Annapolis Water Reclamation Facility<br>7228 Edgewood Rd.<br>Annapolis, MD<br>21403, USA<br>Tel.: 410-222-1938 |
| Moundsville-Glen Dale MWWTP, West Virginia, USA | Hybrid activated sludge; upgrade with Captor | Carbon removal — Nitrification | 8700 m$^3$/d | Influent characterization (average: 1997–1998):<br>- average raw influent flow rate: 7600 m3/d<br>- BOD$_5$: 180 mg/L<br>- liquid temperature: 10°C in winter, 21°C in summer<br>Primary effluent characterization:<br>- BOD$_5$: 115 mg/L<br>- TKN: 20–25 mg/L | Mr. K. Mark Simmons<br>Moundsville-Glen Dale MWWTP<br>800 6th Street<br>Moundsville, WV<br>26041, USA<br>Tel.: 304-845-4360 |

## Table 9.4 Summary of Reported Case Studies of Ammonia Removal at Selected MWWTPs *Contd.*

| Wastewater Treatment Plant and Location | Treatment Technologies | Design Capabilities | Design Average Flow Rate | Remarks | Contact Person |
|---|---|---|---|---|---|
| | | | | Effluent requirements:<br>- no data available<br>Effluent characterization (average: 1997-1998):<br>- ammonium: <2 mg/L (year-round ammonia removal)<br>Configuration:<br>- Primary treatment:<br>- two mechanical bar screens<br>- one aerated grit removal chamber<br>- two primary clarifiers<br>- Secondary treatment:<br>- two activated sludge basins with two tanks in series in each basin:<br>- 1st tank: Captor tank with 10 million sponges and screens to contain media and air knife to prevent screen clogging by sponges<br>- 2nd tank: activated sludge tank with airlift mixed liquor recirculation to Captor tank<br>- RAS<br>- two secondary clarifiers<br>- Sludge treatment: Anaerobic digestion (one primary and one secondary digester)<br>Costs:<br>- treatment plant costs: not available<br>- O&M: annual cost for adding sponges (1 million sponges/2 years): \$6,200 [1]/year<br>Sponge addition is required to replenish the system for the loss of surface area due to the wear of the corner of the sponges. | |
| Lillehammer Sewage Treatment Plant, Norway | MBBR with Kaldnes media | Carbon removal —<br>Nitrification —<br>Denitrification | 26 000 m$^3$/d | Raw influent characterization (average: 1/96 to 12/98):<br>- raw influent flow rate: 14 000 m$^3$/d<br>- BOD$_7$: 121 mg/L<br>- suspended solids: 173 mg/L<br>- total nitrogen: 29 mg/L<br>- total phosphorus: 4 mg/L<br>- temperature range:<br>- min. and max. monthly average: 6.0°C to 12°C<br>- min. and max.: 3°C to 13°C (winter and summer)<br>Effluent requirements:<br>- BOD$_7$: 10 mg/L (annual average), 20 mg/L max. effluent concentration<br>- total nitrogen: 70% removal<br>- total phosphorus: 0.25 mg/L (annual average), 0.5 mg/L max. effluent concentration<br>Effluent characterization (average: 1/96 to 12/98):<br>- BOD$_7$: 4 mg/L<br>- suspended solids: 20 mg/L | Mr. Steiner Bungum<br>Senior Engineer<br>Lillehammer Kommune<br>Storgt. 51<br>P.O. Box 955<br>N-2601 Lillehammer<br>Norway<br>Tel.:<br>011 47 (61) 25 45 40<br>E-mail:<br>sbu@lillehammer.kommune.no |

## Table 9.4 Summary of Reported Case Studies of Ammonia Removal at Selected MWWTPs *Contd.*

| Wastewater Treatment Plant and Location | Treatment Technologies | Design Capabilities | Design Average Flow Rate | Remarks | Contact Person |
|---|---|---|---|---|---|
| | | | | - total nitrogen: 5.8 mg/L (year-round ammonia removal)<br>- total phosphorus: 0.14 mg/L<br>- effluent characterizations are with a combined pre- and postdenitrification<br>Configuration:<br>- Primary treatment:<br>  - two screenings<br>  - three grit chambers<br>  - four primary clarifiers<br>- Secondary treatment:<br>  - two parallel MBBRs with Kaldnes media (K1 type), each reactor includes:<br>    - nine cells in series with mixing and aeration equipment to provide the following flexibility:<br>      - cells 1 to 3 and cells 6 to 8 anoxic or aerobic<br>      - cells 4, 5 and 9 aerobic only<br>  - nitrates can be recycled from cells 5, 6 or 7 to the first cell<br>  - MBBR can be operated with predenitrification, postdenitrification or a combination of pre- and postdenitrification<br>  - lime can be added for alkalinity in cells 5, 6 and 7 to ensure pH level and sufficient alkalinity for nitrification<br>  - external carbon source (ethanol) can be added to cells 6, 7 and 8<br>  - media fill volume fraction of the reactors is 65% of the cells<br>  - end of cell sieves prevents the media discharge, and spiral roll pattern is created with aeration or mixing equipment to avoid media clogging on the sieve<br>  - no RAS is required<br>  - three trains with four flocculation chambers per train to add polyaluminum chloride for phosphorus removal and flocculation of biological solids<br>  - six secondary clarifiers<br>- Sludge treatment:<br>  - two gravity thickeners<br>  - three centrifuges<br>Costs:<br>- upgrade of the plant for nitrogen removal: $9.5 million, compared with an alternative proposal with activated sludge process with a cost of $11.9 million[1]<br>- O&M: $300,000/year,2 based on electricity only (lighting, heating and ventilation of the building)<br>- approximately 10 000–15 000 kg/year of lime is used and 220 $m^3$/year of ethanol is used; no cost on these two components. | |

## Table 9.4 Summary of Reported Case Studies of Ammonia Removal at Selected MWWTPs *Contd.*

| Wastewater Treatment Plant and Location | Treatment Technologies | Design Capabilities | Design Average Flow Rate | Remarks | Contact Person |
|---|---|---|---|---|---|
| Skyway Water Pollution Control Plant, Burlington, Ontario | Conventional activated sludge | Carbon removal — Nitrification | 93 000 $m^3/d$ | | |
| North Tahoe-Truckee MWWTP, USA | Ion exchange (clinoptilolite; regenerant recovery by stripping) | Ammonia removal | 19 000 $m^3/d$ | Raw influent characterization (1993):<br>- average raw influent flow rate: 19 000 $m^3/d$<br>Effluent requirements:<br>- total ammonia-nitrogen limit was at 2 mg/L but had been increased to a higher value due to a revised groundwater discharge<br>Effluent characterization:<br>(before 1993): 2.5 mg total ammonia-nitrogen/L at ion exchange effluent and further reduced to 0.3 mg/L at the breakpoint chlorination stage<br>(1993): 5–6 mg total ammonia-nitrogen/L at ion exchange effluent without breakpoint chlorination<br>Configuration:<br>- four clinoptilolite ion exchange columns<br>- closed-loop ammonia stripping process to recover the regenerant<br>- off-gas absorption in sulphuric acid solution to form ammonium sulphate for fertilizing value<br>- breakpoint chlorination, discontinued utilization<br>Costs:<br>- capital costs: not available<br>- O&M costs: not available | |

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[1] U.S. currency conversion factor for Canadian currency: 1.55.

[2] Electricity costs are based on $0.10/kWh.

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# Appendix A (Supplemental Information on Ammonia Removal Processes)

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To complement the descriptions of the various ammonia removal technologies that were provided in Chapter 5, the following represents useful additional information, such as process characteristics, process design and operational considerations and associated advantages and limitations. This supplemental information is listed in the same order as the process-specific descriptions presented in Chapter 5.

This supplemental information is followed by a summary of commercially available biological and physical/chemical ammonia removal processes, along with some of the respective process vendors and corresponding contact persons (Tables A4, A5, A6 and A7). ***The listing of processes and vendors does not represent an endorsement or recommendation but rather is a summary of the major suppliers, in particular those from whom information was readily obtained.***

## A.1 Suspended-Growth Nitrification Processes

### A.1.1 *Traditional Activated Sludge Processes*

Table A1 provides a summary of design standards in Ontario for conventional and extended aeration activated sludge plants, for both non-nitrifying and nitrifying conditions (OMOE, 1984). Nitrifying activated sludge plants are considered to be low- to medium-rate biological processes depending on the SRT and F:M conditions applied (WEF, 1990).

Activated sludge plants include preliminary and often primary treatment of the raw wastewater prior to its introduction into the aeration tank(s) for biological treatment. Primary clarification is rarely used in extended aeration systems. The influent either can all be fed to the front of the aeration stage or alternatively can be introduced in a step feed arrangement to a number of points along the length of the aeration tank(s). Furthermore, the aeration tank(s) may contain an initial selector zone (e.g., a mixed anoxic compartment(s)) where influent and recycled RAS are combined, which can enhance sludge settleability.

#### A.1.1.1 Process Considerations

The traditional activated sludge technology is among the most prevalent treatment processes in use for municipal wastewater treatment, including well-proven and successful nitrification applications. However, mechanical secondary treatment plants have relatively high O&M requirements because of the need for skilled and regular operator attention to ensure satisfactory performance and due to the mechanical complexity of the processes and components. Many small communities lack the skilled personnel necessary to properly operate and maintain a mechanical facility such as an extended aeration activated sludge plant. The municipality could consider procuring the services of a contract operations company with qualified personnel (USEPA, 1992).

Extended aeration activated sludge processes are available as preengineered package plants for small-flow applications. Extended aeration activated sludge plants are also used to treat flow rates of 20 MLD and greater (USEPA, 1992). In the case of package plants, it is important to ensure adjustable RAS pumping rate capabilities for proper process control. Because of the long aeration periods in extended aeration activated sludge plants, they may be susceptible to low process temperatures in cold-climate applications. In such cases, package plant designs can include insulated covers over both the aeration tank and secondary clarifier to minimize the rate of heat loss.

A limiting consideration in the sizing and operation of an activated sludge process is its final sedimentation stage, which can be susceptible to upsets due to poor sludge settling characteristics and/or excessive solids loads to the secondary clarification stage and consequential loss of biomass in the effluent (USEPA, 1993). Design and operations at high MLSS concentrations should be avoided to minimize difficulties associated with elevated clarifier solids loadings (USEPA, 1993). There should be provision for reliable and adequate excess sludge handling and disposal capacity. Systems for separate and dedicated thickening of WAS are very beneficial.

The application of a selector zone(s) at the inlet to the aeration tank can mitigate many of the potential sludge settleability difficulties. It is particularly advantageous to employ an up-front anoxic zone (i.e., sized for at least 15–30 minutes of HRT at average design flow) for single-stage nitrification systems, to improve sludge

## Table A1 Summary of Activated Sludge Process Design Guidelines in Ontario (OMOE, 1984)

| Aeration Stage | | | Clarification Stage | | |
|---|---|---|---|---|---|
| **Design Parameter** | **Conventional Activated Sludge** | **Extended Aeration Activated Sludge** | **Design Parameter** | **Conventional Activated Sludge**[4] | **Extended Aeration Activated Sludge** |
| Organic loading (kg $BOD_5/m^3$ per day) | 0.31–0.72<br>[0.31–0.72] | 0.17–0.24<br>[0.17–0.24] | Depth (m) | 3.6–4.6<br>[3.6–4.6] | 3.6–4.6 |
| F:M ratio (kg $BOD_5$/kg MLVSS per day) | 0.2–0.5<br>[0.05–0.25] | 0.05–0.15<br>[0.05–0.15] | Surface overflow rate[5] (L/$m^2$ per second) | 0.41<br>[0.34] | 0.41 |
| Minimum HRT (h; based on $Q_{avg.}$) | 6<br>[6] | 15<br>[15] | Weir loading rate (L/m per second) | 2.9<br>[2.9] | 2.9 |
| RAS (% $Q_{avg.}$) | 25–100<br>[25–100] | 50–200<br>[50–200] | Solids loading rate (kg/$m^2$ per day) | <120[5,6] | <120[5,6] |
| SRT (d) | 4–6<br>[>10[1]] | >15<br>[>15] | | | |
| $O_2$ demand (kg $O_2$/kg $BOD_5$) | 1.0[2]<br>[1.0+4.6][3] | 1.5[2]<br>[1.5+4.6][3] | | | |
| Minimum DO (mg/L) | 2.0<br>[2.0] | 2.0<br>[2.0] | | | |
| Minimum residual alkalinity (mg/L as $CaCO_3$) | –<br>[50] | –<br>[50] | | | |

[ ]: With nitrification
$Q_{avg.}$: average influent flow rate
[1] At 5°C.
[2] kg $O_2$/kg $BOD_5$.
[3] kg $O_2$/kg $BOD_5$ + kg $O_2$/kg TKN.
[4] With chemical addition to mixed liquor for phosphorus control.
[5] At peak overflow rate.
[6] Including 100% RAS rate.

settleability by reducing the potential for growth of many types of filamentous organisms.

Larger anoxic zones (e.g., 20–40% of the total aeration tank volume), with provision for mixing but without the entrainment of DO, can be located upstream of the aerobic nitrification zone to achieve predenitrification (WEF, 1998). These combination single-sludge nitrification-denitrification processes typically employ internal recirculation of mixed liquor from the oxic zone to the anoxic zone. Alternatively, process designs and operational strategies can be employed that permit the on/off cycling of aerators throughout the aeration tank (i.e., "on/off aeration process" to create an alternating sequence of aerobic and anoxic conditions in the aeration tank mixed liquor). The single-sludge nitrification-denitrification process can be readily retrofitted into existing activated sludge plants to permit enhanced nitrogen removal.

The following benefits are associated with combined nitrification-denitrification processes (WEF and ASCE, 1992; USEPA, 1993; WEF, 1998):

- improved control of total nitrogen for improved effluent quality;

- reduction in oxygen demand and corresponding aeration energy (e.g., by 10–20%, according to CG&S and Hydromantis, Inc., 1996);
- recovery of alkalinity (e.g., about half of the alkalinity consumed in nitrification is recovered through denitrification);
- improved sludge settleability by reducing bulking potential; and
- reduced potential for floating sludge in the secondary clarification stage.

An important consideration associated with the more complex biological nutrient removal process designs is the reduced aeration volume due to the requirement for up-front anaerobic and anoxic zones. In these cases, the minimum design SRT required for nitrification should be based only on the aerated mixed liquor volume. However, benefits associated with biological nutrient removal processes include reduced aeration requirements due to cBOD removal in the anaerobic and anoxic zones. An overall aeration energy cost recovery of about 30% can be expected in biological nutrient removal processes (Randall *et al.*, 1992). Reduced total sludge production can be expected relative to chemical precipitation for total phosphorus control. Although sludge settleability characteristics can be improved due to contacting in the anaerobic zone and anoxic zones, the anaerobic zones of biological nutrient removal processes can sometimes cause foaming difficulties. Biological nutrient removal processes usually require additional tankage, advanced process control improved sludge handling.

### A.1.1.2 Advantages/Limitations

The activated sludge process is a well-proven wastewater treatment technology with numerous full-scale applications and successful experiences for consistent nitrification. Process control is possible through adjustments of activated sludge age and inventory, DO concentration and degree of mixing to adjust for changes in the environmental conditions and specific nitrification rates. Other benefits include the potential for advanced single-sludge biological nutrient removal process configurations.

Potential process limitations include:

- susceptibility to episodes of poor sludge settling characteristics and foaming, which can adversely affect activated sludge retention, process performance and effluent quality;
- susceptibility to floating sludge losses due to potential denitrification reactions in the secondary clarification stage; and
- uncontrolled losses of biomass, which can reduce the degree of control over sludge inventory and critical SRT for nitrification.

Other variations of the nitrifying activated sludge process with unique respective features are listed below. These processes are often supplied by process and equipment vendors in contrast to more traditional activated sludge processes, which are typically designed by engineering consultants.

## A.1.2 Oxidation Ditch

### A.1.2.1 Process Considerations

Advanced configurations involving multiple interconnected oxidation ditches offer further process flexibility (USEPA, 1993). The Bio-Denitro™ system comprises two tanks working in an alternating mode of operation. One tank operates under anoxic conditions and is fed with raw wastewater and RAS to ensure denitrification. The other tank is fed with mixed liquor from the anoxic tank and operated under aerobic conditions to achieve nitrification. The function of the two tanks is periodically interchanged and flow directions are reversed. It is possible to easily vary the relative denitrification and nitrification capacity of the plant by running both tanks at either anoxic or aerobic conditions. Durations of these various phases are adjustable (WEF, 1998).

Another cost-effective variation of the oxidation ditch technology is the Triple-Ditch process. It consists of three interconnected oxidation ditches and incorporates an operating strategy referred to as phased isolation ditch technology. The ditches alternately serve as oxic, anoxic or quiescent reactors to perform controlled nitrogen removal without the need for external secondary clarifiers or an internal mixed liquor recycle stream. The process provides flexibility in varying the effective volumes allocated for aerobic treatment and settling. Influent flow diversion is controlled by a distribution chamber equipped with three motor-operated weirs. Process control is provided by a programmable logic controller. Automatic DO control is employed to enhance process performance and reduce energy requirements and O&M costs.

### A.1.2.2 Advantages/Limitations

Some of the advantages of the oxidation ditch technology for nitrification include the following:

- excellent treatment performance and high reliability;
- operational simplicity;
- process flexibility;
- well-proven and most suited for low flow rate applications;
- does not normally employ a primary treatment stage;
- improved sludge settleability due to anaerobic and/or anoxic zones;
- reduced sludge yields;
- energy-efficient biological nitrogen removal;
- potential for enhanced biological nutrient removal capabilities; and
- relatively low initial capital cost.

Owing to the extended aeration operating mode, the process may be susceptible to adverse low-temperature effects. In addition to negative effects on treatment performance, the aerators may suffer from freezing in cold-climate applications. Another potential shortcoming includes the relatively high maintenance requirements for the aerators.

## A.1.3 Sequencing Batch Reactor

### A.1.3.1 Process Considerations

While all process phases occur in the same reactor, two or more parallel reactors are essential for treatment unless a very large holding basin is available for storage and equalization of the raw wastewater supply (Randall *et al.*, 1992; WEF, 1998). Advanced variations of the SBR technology include the Cyclic Activated Sludge System (CASS), which is configured with a selector reactor and employs mixed liquor recirculation, and the Intermittent Cycle Extended Aeration System, which incorporates a continuous inflow of influent into an anoxic selector zone during all phases of reactor operations (USEPA, 1993).

The SBR process permits a relatively high degree of process control by adjustment of the relative cycle times of the various phases and is capable of biological nutrient removal operations, depending on the applied conditions and the type of mixing process installed. Process control is provided by a programmable logic controller. Since individual reactors are operated in batch mode, a high initial F:M ratio is experienced while filling the reactor containing settled biological solids with raw wastewater. During this phase, the reactor serves as a selector zone to prevent the growth of filamentous organisms. However, the process may be somewhat susceptible to sudden shock loads, although this will be dampened by the large inventory of process biomass and relatively large reactor volumes.

High levels of treated effluent quality are possible with the SBR technology, including effective nitrification, depending on the prevailing process conditions. SRT control for nitrification should be based on the fraction of time the process biomass is maintained under aerobic conditions. A recent survey of 75 SBR plants in North America revealed residual $BOD_5$, TSS, total nitrogen and total ammonia-nitrogen levels of 6 mg/L, 9 mg/L, 4 mg/L and 1.5 mg/L, respectively (Hydromantis, Inc. and SBR Technologies Inc., 1998).

SBR processes are typically designed and marketed by equipment suppliers, unlike more traditional treatment processes, which are usually designed by engineering consultants. Additional details on SBR process designs, operational considerations, optimization measures and full-scale experiences can be found in Hydromantis, Inc. and SBR Technologies Inc. (1998).

### A.1.3.2 Advantages/Limitations

Some of the benefits of the SBR technology include the following:

- relatively high degree of process reliability, flexibility and control over process conditions and cycle times for microorganism selection and effluent quality;
- operational simplicity, with low O&M requirements and costs;
- does not normally employ a primary treatment stage;
- does not require an external secondary sedimentation stage and sludge recycle system;
- batch operations are suited for wide variations in flow rates experienced in smaller communities (USEPA, 1992);
- improved sludge settleability due to anoxic conditions during react/fill phase;
- reduced sludge yields;
- relatively inexpensive capital cost versus conventional continuous-flow activated sludge processes (Hydromantis, Inc. and SBR Technologies Inc., 1998);
- relatively energy efficient;
- can be designed for and operated in biological nutrient removal mode, including biological phosphorus removal; and
- jet aeration permits independent control of mixing and aeration, which may be important for biological nutrient removal operations (USEPA, 1992).

Potential process limitations or operational problems include:

- some difficulties reported with the effluent decant system;
- susceptibility to shock loads such as the periodic supply of landfill leachate, which may cause upsets to the nitrifying microorganisms; equalization and controlled additions of potential inhibitory streams are recommended;
- discontinuous discharges can cause difficulties for downstream processes such as continuous sand filtration and effluent disinfection; and
- lack of interconnections between reactors to permit transfer of mixed liquor, if necessary.

## *A.1.4 Membrane Bioreactor*

### A.1.4.1 Process Considerations

Aeration from the base of the immersed membrane modules is essential to control against excessive fouling of the filtration membranes. Furthermore, in the case of the immersed hollow fibre membrane system, regular cleaning procedures must be employed to sustain reasonable filtrate flux levels. This includes frequent backpulsing with filtered effluent (i.e., reverse pumping of permeate from inside to outside of the hollow fibres), daily backwashing with a sodium hypochlorite solution and occasional removal of the membranes for extensive chemical cleaning in an external tank (WERF, 2000). Care must be exercised during in-place cleaning of the membranes with chlorine chemicals to prevent inhibition of the relatively sensitive nitrifying process biomass.

Although the MBR process does not require a primary treatment step, it does require fine screening (i.e., 2 mm) of the wastewater supply. This is to prevent problematic clogging of membranes with suspended contaminants such as hair and fibre. As noted, an anoxic zone at the inlet to the aeration tank is typically employed wherein the pretreated influent and recirculated mixed liquor are combined and denitrification can be achieved.

To sustain the concentrated nitrifying biomass in the compact MBR process, high-efficiency fine-bubble aeration is required to satisfy high oxygen transfer requirements. It has been reported that at elevated MLSS concentrations, oxygen transfer rates may become limited and can affect nitrification (Fernandez *et al.*, 2000). The aeration provided by the coarse-bubble aerators utilized for membrane scouring has minimum oxygen transfer efficiency.

Depending on design flux levels, the membrane filtration process could potentially pose as a hydraulic bottleneck, particularly at high sustained peak influent flow rate conditions. It would likely be more economical to install upstream or in-tank flow equalization than to design the membrane system for peak flow conditions (Crawford *et al.*, 2000; Fernandez *et al.*, 2000).

The application of MBR technology for enhanced treatment of municipal wastewaters can involve greenfield installations of covered modular treatment plants or, alternatively, the retrofitting of existing activated sludge process aeration tanks with membranes for *in situ* filtration of mixed liquor. Plant retrofits would involve:

- insertion of membrane modules into open aeration tanks;
- piping the membrane module frames to the filtrate removal pumps and clean-in-place system;
- installation of external membrane cleaning tanks;
- installation of additional blowers if increased capacity is required;
- retrofitting the main aeration system to a fine-bubble aeration, if necessary; and
- conversion of existing secondary clarifiers into primary clarifiers and/or flow equalization tanks.

### A.1.4.2 Advantages/Limitations

The MBR technology can produce very high quality treated effluent (i.e., suspended solids free and the potential for virtually complete removal of total ammonia-nitrogen). The process is still considered a developing technology, although it is well demonstrated on the very small to small scale for treatment of domestic wastewaters. However, medium- to large-scale applications (i.e., >5 MLD) are still relatively recent, with limited available performance information and reported experiences (WERF, 2000).

Some of the MBR process advantages are:

- potential for upgrading existing activated sludge plants with membrane retrofits for enhanced capacity and nitrification;

- high degree of SRT control due to absolute retention of process biomass; and
- low sludge yields at the elevated SRTs.

Some potential process limitations include:

- relatively high capital cost for the advanced MBR process with superior effluent treatment capabilities;
- elevated energy consumption for coarse-bubble aeration required for scouring of filtration membrane surfaces;
- the requirement for periodic replacement of membranes, which can be costly;
- potential for filtration system to pose as a hydraulic bottleneck, depending on design flux levels;
- need for adequate flow equalization to be designed into the system to dampen peak flow rates to prevent problematic rapid increases in transmembrane pressures (i.e., caused by excessively high filtrate flux levels) that would require more frequent and rigorous cleaning of membranes outside of the bioreactor (Fernandez *et al.*, 2000); and
- potentially high maintenance requirements and costs for membrane cleaning, which may cause particular concerns and uncertainties for relatively large-scale applications (WERF, 2000).

## A.1.5 Aerated Lagoon Processes

### A.1.5.1 Process Considerations

Nitrification of ammonia can occur in aerated lagoons and possibly in aerated facultative lagoons, but only at warm temperature conditions and at low loading rates. The degree of nitrification that can be achieved is dependent on the system HRT and the prevailing process temperature, DO and mixing conditions (Environment Canada, 1987; USEPA, 1993). Nitrification activity can be severely limited or lost during cold-climate operating conditions. The low-rate design of lagoon processes makes them particularly susceptible to low temperature conditions and associated negative effects on biological treatment performance.

Aerated lagoon and aerated facultative lagoon processes applied in cold-climate locations require increased treatment time and/or effluent storage capacity (Environment Canada, 1987; NovaTec Consultants Inc., 1996). To achieve target total ammonia-nitrogen concentrations of less than 5 mg/L in the effluent discharge, lagoon processes require adequate storage capacity in a downstream facultative/storage cell(s) to allow for enhanced treatment and controlled annual discharge of acceptable quality effluent during autumn (USEPA, 1992; NovaTec Consultants Inc., 1996). As a secondary benefit associated with the provision for effluent accumulation and storage, operational costs are lower than for continuous-discharge lagoons because of a reduction in process operation, monitoring and analysis requirements. Alternatively, aerated lagoon and aerated facultative lagoon-based MWWTPs could be upgraded with an add-on nitrifying process (NovaTec Consultants Inc., 1996). Technological options for the post-treatment of lagoon effluents to achieve year-round nitrification in cold-climate applications are detailed in Section 6.1.3.

Additional information on design and operational aspects, full-scale experiences and performance characteristics with aerated lagoon and aerated facultative lagoon processes can be found in USEPA (1983, 1992), Metcalf & Eddy (1991), OMOE and Environment Canada (1993) and NovaTec Consultants Inc. (1996).

### A.1.5.2 Advantages/Limitations

Aerated lagoons and aerated facultative lagoons represent an economical alternative for municipal wastewater treatment where low-cost land is readily available. Lower levels of operator skill and O&M are required for lagoon systems than for the higher-rate mechanical treatment processes described above. Aerated lagoons offer a greater degree of process control than do the more conventional facultative lagoons. However, energy consumption and operating costs for mechanical aeration are greater than for conventional lagoons that rely mainly on algal growth and photosynthetic activity for oxygenation.

Some of the limitations associated with aerated lagoon processes include the following:

- very limited total ammonia-nitrogen removal capability during conditions of low process temperature that are typically experienced in cold-climate applications in Canada;
- in addition to significant temperature effects, nitrification potential is strongly dependent on process HRT and degree of mixing;
- aerated facultative lagoon processes have a reduced potential for nitrification owing to incompletely mixed conditions in the aerated cell; however, some nitrification may occur in the downstream facultative zone;
- requirement for long-term effluent storage capacity to permit controlled seasonal or annual discharges in cold-climate applications or, alternatively, the requirement for an add-on nitrifying process stage; and
- the impracticality of the long-term effluent storage option for relatively large flow, cold-climate applications; however, the add-on treatment process option could be considered to attain year-round nitrification in continuous-discharge mode.

## A.2 Attached-Growth Nitrification Processes

### A.2.1 *Trickling Filters*

### A.2.1.1 Process Characteristics

**Media**

Rock, slag and other natural media have been widely used in early trickling filter installations. Alternative materials, such as plastic and wood media, were developed and introduced in standard practice to increase the surface area per unit of volume, improve ventilation characteristics and reduce clogging. Plastic media can be fabricated in different structural configurations, such as corrugated sheet assemblies, small balls or rings, and plastic strips. Corrugated sheet assemblies are stacked in the filters, plastic balls or rings are disposed in a random configuration, and plastic strips are suspended from a support structure on top of the filter. Considering their advantages, plastic media are now typically selected for new plant design or existing filter upgrade. Table A2 presents comparative characteristics for different types of media used in trickling filters.

Corrugated plastic sheets can be further divided into vertical and cross-flow configurations. Vertical media have a configuration that orientates the flow downward. Cross-flow media have ridged corrugated sheets set at 45- or 60-degree angles to each other. It has been shown that the cross-flow configuration provides higher HRT between the biofilm and the influent and better oxygen transfer efficiency.

The properties of the media that are of interest for the performance of trickling filters include the specific surface area and the void space. The potential for a larger biomass film development per volume of media increases with a higher specific surface area. The increase of the void space percentage in the media permits the handling of higher hydraulic loadings and favours

**Table A2 Comparative Physical Properties of Typical Trickling Filter Media (USEPA, 1993; WEF, 2000b)**

| Media Type and Characteristics | Unit Weight ($kg/m^3$) | Specific Surface Area ($m^2/m^3$) | Void Space (%) |
|---|---|---|---|
| Rock (50–100 mm) | 1600 | 46 | 60 |
| Rock (30–80 mm) | 1400 | 62 | 50 |
| Wood (1220 × 1220 × 48 mm) | 165 | 46 | N/A [1] |
| Plastic sheets, cross-flow, corrugated | 24–45<br>24–45 | 98<br>138 | >95<br>>95 |
| Plastic sheets, cross-flow, corrugated, 60-degree angle | 24–45<br>27–63<br>24–45 | 138<br>157<br>223 | 95<br>95<br>95 |
| Random pack media, polypropylene | 53<br>67<br>99 | 125<br>157<br>279 | 95<br>93<br>90 |
| Random pack media | 53<br>61<br>72 | 102<br>144<br>180 | 94<br>93<br>92 |
| Flexible hanging plastic sheets | N/A<br>N/A<br>N/A | 98<br>138<br>223 | N/A<br>N/A<br>N/A |

[1] N/A = Not applicable.

better oxygen transfer. As shown in Table A2, plastic media typically provide substantially higher specific surface areas and void space when compared with rock media, which enable plastic media to handle higher organic and hydraulic loadings.

The plastic media specific surface area currently used for combined $BOD_5$ removal and nitrification ranges from 88 to 105 $m^2/m^3$ (surface of media/volume of media). The specific surface area for nitrification only or tertiary nitrification is typically 135–150 $m^2/m^3$ (WEF, 2000b). However, if fine primary screening and biomass accumulation control is used, media specific surface areas of between 135 and 145 $m^2/m^3$ may be used for $BOD_5$ removal, as the clogging potential is reduced (WEF, 2000b).

The use of high-density media in the range of 200–240 $m^2/m^3$ is not recommended for tertiary nitrification, as patchy biofilm (dry spots) may develop and reduce the nitrification rates achievable (WEF, 2000b). Moreover, it is difficult to completely wet the surface of high-density media with a specific surface area higher than 150 $m^2/m^3$, and such media are more subject to clogging.

The depth of the trickling filters filled with rock media typically ranges from 1.2 to 3.0 m due to limited ventilation (USEPA, 1993). Plastic media trickling filters have typical depths from 4.5 to 12 m (WEF, 1998), considering the lower weight of plastic media compared with rock media and the better ventilation properties. The advantage of higher depths is the lower land area requirements and the reduction of recycle flow to keep the media wetted. However, higher depth costs more for raw wastewater, and effluent recycle pumping and forced ventilation may be required due to the increased loading per unit floor area. Uncovered trickling filters with plastic media should have the top layer of media protected from ultraviolet deterioration.

### Combined BOD$_5$ Removal and Nitrification (Single Stage)

The nitrification process is strongly influenced by the organic loading applied onto the filter. In the single-stage configuration, the nitrification zone is typically located in the lower-medium portion of the filter where the growth of autotrophic nitrifiers is favoured following the removal of the organic load by the upper-filter heterotrophic biomass portion and the decreased competition for oxygen between heterotrophs and nitrifiers. If the organic load and strength are increased, the heterotrophic organisms grow faster than the autotrophic nitrifiers, and the nitrification zone is displaced downward. In fact, the nitrifiers become covered by heterotrophic growth in the biofilm beyond the oxygen diffusion depth and cease their nitrification activity at this location of the filter.

The minimum hydraulic surface loading rate applied to trickling filters to achieve nitrification varies with the type of medium and is usually in the range of 1.25–3.0 m$^3$/m$^2$ per hour, based on the plan area of the filter (USEPA, 1993; WEF, 1998). The minimum hydraulic surface loading rate is obtained with the wastewater pumping rate to the filter and with additional treated effluent recirculation, if required. The objective of the hydraulic loading rate is to keep the surface of the media wetted. Proper wetting permits the enhanced utilization of media surface for biofilm development and optimization of the organic loading rates in the trickling filter at all flow conditions.

The range of volumetric organic loading rates typically applied on trickling filters with rock and plastic media to achieve partial or nearly complete nitrification is indicated in Table A3.

The loading rates are influenced by factors such as the hydraulic surface loading rate, recirculation ratio, oxygen availability through efficient ventilation and biomass control, and temperature. Such factors are discussed in the process considerations section.

A surface organic loading rate of 2.5 g BOD$_5$/m$^2$ per day, based on the surface area of the media, is reported to be necessary for 90% total ammonia-nitrogen removal with rock and plastic media (USEPA, 1993). Surface organic loading rates below 1.0 g BOD$_5$/m$^2$ per day are indicated to ensure residual total ammonia-nitrogen concentrations lower than 2.0 mg/L (USEPA, 1993).

The design of single-stage processes uses empirical methodology based on graphical relations established with data available from the various trickling filters operating under this configuration. The kinetics of combined BOD$_5$ removal and nitrification are complex, and a fundamental approach to design has been elusive.

The TKN surface removal rate (g TKN/m$^2$ per day, based on the surface area of the media) required at a given cBOD$_5$:TKN to correctly achieve nitrification can be determined with the use of Figure A1. The graphic is derived from an extensive array of operating results and provides a relationship between influent cBOD$_5$:TKN ratio and the nitrification rate. The influent temperature range used to obtain this graphical relation was 9–20°C. This relation

**Table A3 Volumetric Organic Loading Rates to Single-Stage Process (WEF, 1998)**

| Media Type | 75–85% Nitrification (kg $BOD_5/m^3$ per day)[1] | 85–95% Nitrification (kg $BOD_5/m^3$ per day)[1] |
|---|---|---|
| Rock media | 0.096–0.16 | 0.04–0.08 |
| Plastic media | 0.19–0.29 | 0.08–0.15 |

[1] Loading rates are based on the volume of the media.

indicates that the ability to nitrify is strongly dependent on the organic loading rate. As the organic loading and $cBOD_5$:TKN ratio are increased, nitrification efficiency is reduced due to competition between heterotrophs and nitrifiers for oxygen. The point at which a nitrifying trickling filter is considered as a single-stage process is arbitrarily identified as $cBOD_5$:TKN ≥ 1:1 (WEF, 2000b). The volume of medium required to achieve nitrification can be evaluated from the TKN surface removal rate determined and the specific surface area ($m^2/m^3$) of the medium to be used.

The surface $BOD_5$ loading rate (kg $BOD_5/m^2$ per day) for a required nitrification efficiency (%) can be determined from operating plant data correlations for rock and plastic media, as shown in Figure A2. The volume of medium required to achieve nitrification can be evaluated from the surface $BOD_5$ loading rate determined and the specific surface area of the media to be used. Alternatively, the volumetric organic loading rate can be calculated with the volume of media estimated with TKN surface removal rate and compared with the ranges identified in Table A3.

### Tertiary Nitrification (Two Stage)

Nitrification in a two-stage system requires a secondary clarified influent from a trickling filter or any other biological processes with a low $cBOD_5$:TKN ratio. Tertiary nitrification trickling filters should have $cBOD_5$:TKN ratios in the range of 0.25–1:1 (USEPA, 1993; WEF, 2000b). It has been demonstrated that the secondary influent $BOD_5$ must be lower than 30 mg/L to initiate nitrification and that complete nitrification would be achieved with $BOD_5$ lower than 15 mg/L (WEF, 2000b). The filtered $BOD_5$, referred to as soluble $BOD_5$, should be lower than 20 mg/L to initiate nitrification, and the maximum nitrification rates would occur with secondary influent soluble $BOD_5$ at 4–8 mg/L (WEF, 2000b). The low $BOD_5$ concentration of the secondary influent is typically accompanied by low TSS concentrations.

In the absence of a significant load of $BOD_5$, the nitrification rate becomes highly dependent on the available oxygen and the total ammonia-nitrogen concentration. In the upper portion of the trickling filter, where the total ammonia-nitrogen concentration is high, the availability and transfer of oxygen are the governing factors for the nitrification rate. Due to the low organics level, there is little competition between heterotrophic and nitrifying microorganisms. The nitrification rate under these conditions is referred to as a zero-order reaction rate and is typically estimated as 1.28 g ammonium-nitrogen/$m^2$ per day (USEPA, 1993; WEF, 2000b). A wide range of zero-order ammonium-nitrogen removal rates has been observed, from 1.0 to 3.0 g ammonium-nitrogen/$m^2$ per day (WEF, 2000b); however, these high rates may be achieved only with highly efficient oxygen transfer and improved understanding and optimization of trickling filters.

Nitrification rates change to a first-order rate when the ammonium-nitrogen concentration reaches between 3 and 5 mg/L in the intermediate portion of the trickling filter (USEPA, 1993; WEF, 2000b). At these ammonium-nitrogen levels, the diffusion of ammonium-nitrogen, rather than oxygen, in the biofilm is the controlling factor for the nitrification rate. Under these conditions, total ammonia-nitrogen removal rates decrease with ammonium-nitrogen concentration reductions.

Thus, a graphical representation of ammonia removal rates with the depth of the trickling filter would show a straight line of the removal rate in the upper portion of the filter, as the availability of the oxygen is the controlling factor; a decrease of the ammonia removal rate would start in the lower portion of the filter as the ammonium-nitrogen diffusion becomes the controlling factor.

The minimum hydraulic surface loading rate applied to a second-stage trickling filter to achieve nitrification is reported as 1.96 $m^3/m^2$ per hour (WEF, 1998). Hydraulic surface loading rates of 3.6 $m^3/m^2$ per hour and above have been found to provide the highest ammonium-nitrogen surface removal rates (USEPA, 1993). As for the single-stage process, the hydraulic surface loading rate may vary with the type of medium and should keep the surface of the medium wetted to avoid patchy growth of nitrifiers in the lower portion of the filter, which would lead to lower total ammonia-nitrogen surface removal rates.

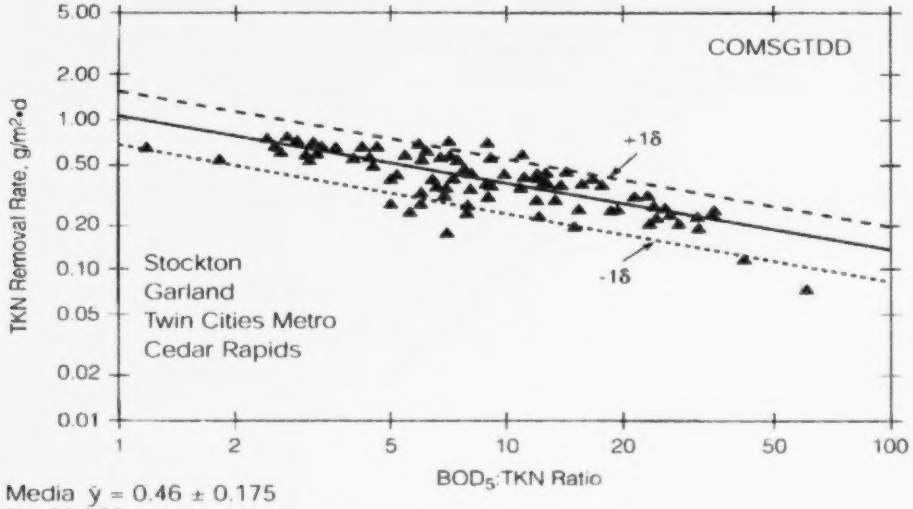
Overall ammonium-nitrogen surface removal rates for the entire depth of a tertiary nitrifying filter are reported as 1.0 $g/m^2$ per day (WEF, 1998) to obtain an effluent ammonium-nitrogen concentration of 3.0 mg/L in a temperature range of 10–30°C, and 0.30 $g/m^2$ per day (USEPA, 1993) to obtain effluent total ammonia-nitrogen of less than 2.0 mg/L in a temperature range of 9–27°C.

As for the single-stage process, the surface removal rates are influenced by factors such as the hydraulic surface loading rate, recirculation ratio, oxygen availability through efficient ventilation and biomass control, and temperature. Such factors are discussed in the process considerations section.

The design of tertiary nitrification processes may use an empirical methodology based on graphical relations established with data available from the various trickling filters operating under this configuration or may be achieved by estimating ammonia removal rate variations within the filter using mathematical models.

The media surface area required to achieve a given effluent total ammonia-nitrogen concentration can be estimated with the use of an empirical relationship such as shown in Figure A3. The graph indicates the major effect of temperature on the total ammonia-nitrogen surface removal rate, with the rate increasing with temperature. The shapes of the curves are typical of the zero- and first-order total ammonia-nitrogen surface removal rates previously described. The straight lines are related to zero-order rates, and the curved lines are related to first-order kinetics. As the level of effluent total ammonia-nitrogen concentration is lowered, the total ammonia-nitrogen surface removal rate decreases, which calls for a higher media surface requirement. The volume of medium required to achieve the level of effluent ammonia is evaluated from the medium surface area determined and the specific surface area of the medium to be used.

**Figure A1** **Correlation Between TKN Surface Removal Rate and Wastewater cBOD$_5$:TKN in Single-Stage Trickling Filters (WEF, 2000b)**



Media $\bar{y} = 0.46 \pm 0.175$
$\bar{x} = 11.081$
$\approx 15\ °C\ TKN_{OX} = 1.086\ (BOD_5:TKN)^{-0.44}$

Mathematical models from Parker and from Albertson and Okey (WEF, 2000b) may be used to design the filter capacity based either on total ammonia-nitrogen removal rate calculation at various points of the filter depth or on estimating the first- and zero-order portions of the media volume. These models are corrected for total ammonia-nitrogen concentration, media ventilation and wetting characteristics and use temperature sensitivity relationships.

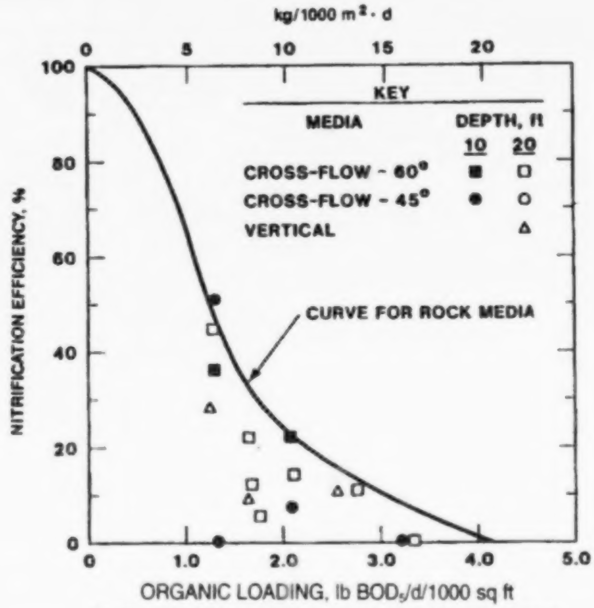
The design relationships and the nitrification predictability for single-stage and tertiary nitrification are empirical and not well defined. Process considerations, mentioned hereunder, should be taken into account to ensure satisfactory operation of trickling filters.

### A.2.1.2 Process Considerations

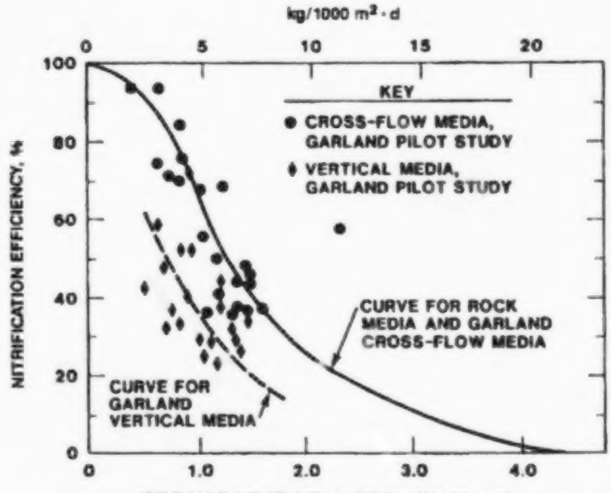
Some key process considerations are particularly important to provide a sustained low total ammonia-nitrogen effluent concentration in single-stage and tertiary nitrification trickling filters. This section briefly covers these process considerations.

Low temperatures can significantly influence the efficiency of the biological treatment process and affect total ammonia-nitrogen removal rate. Thus, temperature has to be taken into consideration in the designing of the trickling filter process. However, the impact of temperature on nitrification process performance is influenced, and in some cases lessened, by other inadequate design factors, such as hydraulic and organic loadings, oxygen availability, ventilation, total ammonia-nitrogen concentrations, wetting efficiency of the media and biofilm control. Studies have shown that in conditions of low temperature, it was excessive biomass growth accumulation, which reduces aerobic surface area, and channelling of water in the media zone,

**Figure A2 BOD$_5$ Surface Loading Rate Effects on Nitrification Efficiency (WEF, 2000b)**

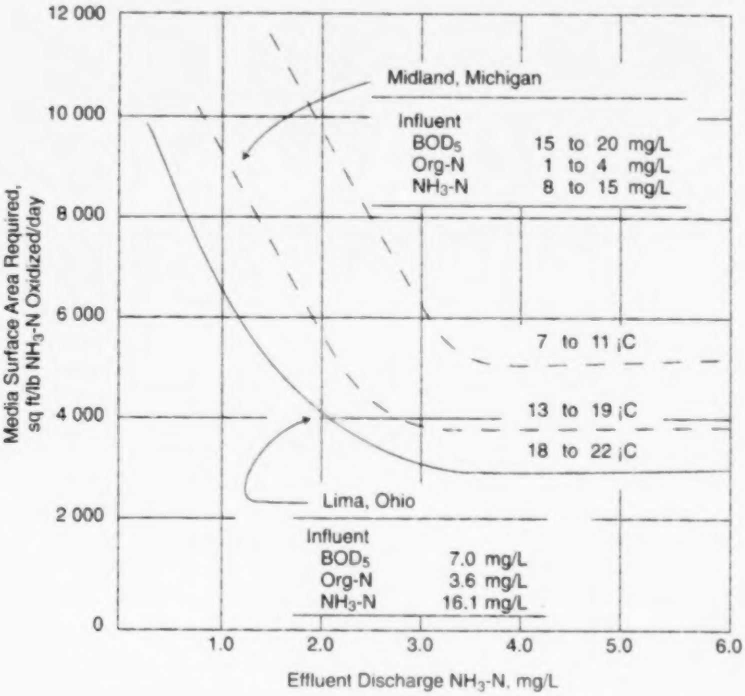


Nitrification efficiency versus organic loading at Atlanta



Nitrification efficiency versus organic loading at Garland

**Figure A3 Tertiary Nitrification Performance with Trickling Filters (WEF, 1998)**



which reduces wetting efficiency, that reduce the performance of the trickling filter. Biomass control improvement under these conditions should be implemented to reduce the temperature dependency and improve filter efficiencies.

Variations of pH within the trickling filter may influence the nitrification performance. Significant pH effects may be reduced or avoided by ensuring effluent alkalinity higher than 50 mg $CaCO_3$/L. In the case of low influent alkalinity, nitrification may proceed to a certain extent in the filter, and pH reduction may occur in the lower portion of the filter only. In the absence of effluent recirculation, pH depression may not be a significant factor. This suggests the use of tall filter towers to minimize recirculation and reduce pH decreases of the filter influent.

Air requirements and ventilation considerations are essential to satisfy the required oxygen to maintain aerobic conditions for biological activity. The amount of oxygen required for cBOD$_5$ oxidation and nitrification of ammonia is approximated as 20 kg/kg oxygen demand and 50 kg/kg oxygen demand, respectively (USEPA, 1993; WEF, 1998, 2000b). Ventilation of trickling filters may rely on natural draft or mechanical draft. Natural draft is induced by the difference in air and wastewater temperatures and the humidity differences between ambient air and air in the trickling filter. This natural draft may provide sufficient aeration; however, periods of small differences in air and wastewater temperatures may result in no airflow through the filter. These periods are reported to occur in the morning and evening hours during the peak oxygen demand. Considering the potential for low oxygen transfer periods associated with natural draft, mechanical ventilation may be required.

Mechanical ventilation can be provided by the use of forced or induced draft fans that ensure a sustained supply of oxygen. The benefits of forced ventilation include the constant oxygen availability to aerobic biofilm biological processes, such as tertiary nitrification, and biological removal of odours by downdrafting and upflowing through a secondary or tertiary tower. Increased process temperatures may be promoted in cold-climate applications. Thus, the performance and operation of the trickling filter processes may be optimized and stabilized.

Recycle loads from solids processing should be returned ahead of primary treatment and taken into consideration as part of the hydraulic and organic loadings to the trickling filters. If anaerobic digester supernatant is available, this ammonium-nitrogen-laden return stream could be stored and recirculated to the filter during low loading periods to favour more consistent ammonia penetration to the lower portion of the filter and avoid patchy biofilm.

The wetting efficiencies of the media surface areas are a characteristic that may vary with the type, density and configuration of the media. Studies have shown that rock and plastic media, including vertical and 45- to 60-degree cross-flow media, had only a fraction of their surface area wetted. High-density surface area media are reported to have poor wetting characteristics, and their use should be limited with improved wetting techniques. The wetting efficiency and HRT are two interrelated factors that can affect treatment efficiency. Studies have shown that cross-flow media configuration has a longer HRT compared with vertical-flow media, which suggests a better wetting efficiency for the cross-flow media. The best results with trickling filters are obtained with the maximum wetting of the media and the aerobic biofilm surfaces. The wetting efficiency is improved with recirculation of the treated effluent and the influent dosing practices on top of the media.

Recirculation of treated effluent permits the increase of hydraulic loading rates, which improves influent distribution, enhances the surface area wetting efficiency and increases aerobic surface area. A higher hydraulic rate helps to maintain a shear force to slough excess biomass growth and reduce clogging effect. Recirculation may dampen variations in loadings applied to the filter. A recirculation ratio primary selection of 1:1 is considered appropriate (USEPA, 1993). As shown in Figure 5.6, the recirculated flow may originate from the filter or the secondary clarifier effluent and be pumped upstream of the trickling filter or the primary treatment stage. The recirculated effluent flow pattern should be taken into consideration in the hydraulic capacity of the primary settling tanks and the trickling filters. The benefits of recirculation can also be achieved to a certain degree by reduction and control of the influent distributor speed.

The influent distributor operation and the dosing rate may directly affect the operation and the efficiency of the trickling filter. The dosing intensity is described by the SK factor, which represents the depth of water deposited during the passage of one distributor arm. The usual distributor operation has a rotational speed of 0.5–2 minutes per revolution, a dosing frequency to the filter of 10–60 seconds with a two- or four-arm distributor, and a related SK value of 2–10 mm per pass of the distributor arms (WEF, 2000b). Studies have shown that reducing the speed of the rotary distributor at 1.3–45 minutes per revolution and increasing SKs to 14–720 mm may enhance the efficiency and capacity of the trickling filter (WEF, 2000b). A proprietary approach to controlling SK is to vary the SK as an inverse function of the influent loading. Low SK values are then applied during high loading periods and provide longer retention time for treatment, and high SK values are applied at low loading periods and provide an intermittent biofilm sloughing cycle. Preliminary recommendations for SK values at an average volumetric loading rate of 0.25 kg $BOD_5/m^3$ per day are 10–30 mm per pass at high loading periods and ≥200 mm per pass at low loading periods (WEF, 2000b).

The reduction of distributor speed and the increase of the operating range of SK values provide several advantages. Sloughing of excess biomass growth on the surface of the medium is improved with high SK, thus reducing the biofilm thickness. Wetting of the media and oxygen transfer efficiencies are increased with higher dosing intensities, thus reducing patchy biofilm and dry spots throughout the filter depth. Ammonium diffusion within the aerobic biofilm may be improved. Nitrifying biomass may then develop deeper in the lower zones of the filter and enhance the media effectiveness. Consequently, more stable performance and higher nitrification rates have been observed.

Optimum SK values may be site and process specific. Thus, the influent distributor should offer a wide range in SK operating values to allow the operators to determine and apply the best operating modes. Electrical radial distributors with timers and variable-speed drives can more reliably offer the degree of control to cover large SK values compared with hydraulic propulsion and fixed-nozzle arrangements.

### A.2.1.3 Advantages/Limitations

Trickling filters have historically been operated for many years, thus providing a better understanding of their operation and appurtenances required to maintain treatment efficiency. Trickling filters may achieve combined $BOD_5$ removal and nitrification and tertiary nitrification. Proper design procedures and operation permit the achievement of trickling filter performance equalling that of suspended-growth systems (WEF and ASCE, 1998). The major advantages related to trickling filters are outlined as follows:

- Treated effluent quality is comparable to activated sludge effluent.
- Nitrifying trickling filters can produce effluent with total ammonia-nitrogen lower than 5 mg/L.
- Land area for trickling filters is typically smaller than for activated sludge treatment.
- Plastic media with higher specific surface area and void space than rock media are commercially available and permit the handling of higher hydraulic and organic loadings and favour better oxygen transfer.
- Plastic media permit trickling filters with depth ranging from 4.5 to 12 m due to the low weight of the media, thus reducing the footprint of the filter and effluent recirculation for wetting of the media surface.
- Recent improvements in adequate hydraulic application modes and air-forced mechanical ventilation can help to provide better trickling filter performance.
- Electrical radial influent distributor operation with variable-frequency drive improves the hydraulic loading application by adjustment of

the rotation speed (minutes per revolution) and the dosing intensity (SK factor).

In terms of limitations, trickling filters are affected by the necessity to maintain sufficient hydraulic loading on the filter to ensure wet conditions on the surface of the medium and by the accumulation of excess biofilm growth. Limitations include the following:

- Optimal wetting of the medium surface area is required to maximize the biofilm surface area and the diffusion in the biofilm of DO and ammonium.
- Patchy biofilm and dry spots reduce the trickling filter nitrification performance and may be caused by poor wetting efficiency from insufficient hydraulic loading and too high density medium (>150 $m^2/m^3$). Improved wetting efficiency and biofilm growth surface area may be obtained with recirculation of the treated effluent and upgrade of influent application modes, i.e., dosing rates and intensities (SK values).
- Excess biofilm growth accumulation may clog the void space of the medium, thus severely reducing the oxygen and nutrient diffusion in the biofilm and creating a preferred flow channel. Sloughing of excess biomass growth may be improved with daily control of influent application and dosing intensity.
- Influent application modes and dosing rates and intensities, i.e., operating range of SK values, are site and process specific.
- Hydraulic propulsion and fixed nozzle arrangements for influent distribution do not provide a high degree of control of the dosing rates and intensities.
- Natural draft ventilation may be insufficient to provide the required oxygen, and potential odour problems may be experienced.

Further information on operational problems and possible corrective actions related to trickling filters is provided in Section 7.2.2.

## A.2.2 *Rotating Biological Contactors*

### A.2.2.1 Process Characteristics

**Media**

RBC media typically consist of polyethylene discs or spiral wound sheets. The media have a surface configuration that maximizes the surface area and promotes turbulence and flow circulation around the biofilm. Standard RBC units are 3.5 m in diameter and 7.5 m in length. The standard medium has a surface area of 9300–9600 $m^2$ per shaft, while a high-density medium has a surface area of 13 900–14 500 $m^2$ for a similar shaft size (USEPA, 1993; WEF, 2000b). Media should be protected from ultraviolet deterioration, and covers should be provided in cold climates to prevent heat loss and ice formation.

Surface media density is a major factor to ensure that the required biofilm is provided and the surface removal rates are optimized according to the type of treatment. The standard-density medium is used mainly in the organics removal passes where the biofilm growth is thick. The standard medium offers wider passageways, avoids or reduces clogging problems and permits circulation of wastewater through the medium. High-density media offer narrower passageways, and their application is more limited to thin biofilm growth conditions that can be expected in the final passes of the single-stage process and in the tertiary nitrification process.

**Combined $BOD_5$ Removal and Nitrification (Single Stage)**

Influent wastewater circulation through the series of rotating contactor passes leads to the development of a biofilm as a function of substrate availability. Thus, the biomass in the first passes is composed mainly of heterotrophic organisms associated with $BOD_5$ removal. The organic load applied in the first pass is high, and the biofilm growth is typically thick. As the $BOD_5$ load is reduced along the passes, the biofilm thickness shrinks. The reduction of the $BOD_5$ load and the $cBOD_5$:TKN ratio to a critical low level permits the nitrifiers to initiate nitrification activity and remove the total ammonia-nitrogen load. In such a case, oxygen is available to nitrifiers, as the growth of heterotrophic organisms is limited by a low organic substrate load.

In a single-stage application, the total surface area of the medium required to achieve complete treatment is the sum of the medium surface area for organic removal and for nitrification. If insufficient medium surface is provided in the first organic removal passes, organic removal will occur in later passes, and nitrification will be shifted towards the final passes or may fail.

The installation of rotating contactors in a series of passes is required to achieve low effluent $BOD_5$ and total ammonia-nitrogen concentrations. Manufacturers usually recommend the number of passes as a function of the organic and total ammonia-nitrogen removal efficiency or treated effluent concentration targets.

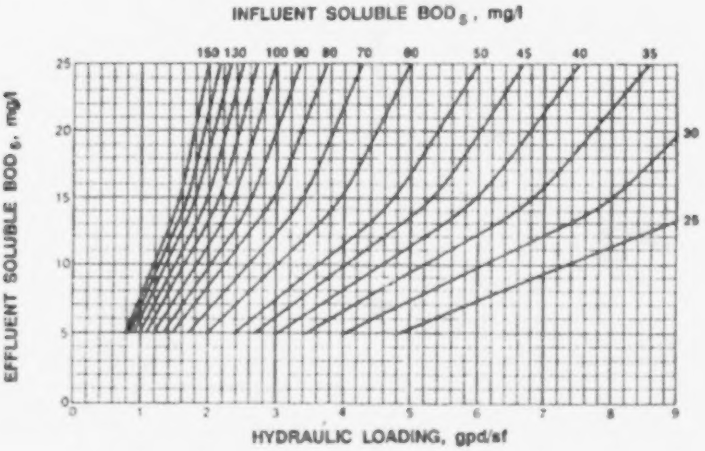
The maximum surface organic removal rates for $BOD_5$ load removal in the first passes of the RBC system are 12–14.5 g soluble $BOD_5/m^2$ per day and 24–29 $BOD_5/m^2$ day, based on the surface area of the medium, assuming a 50% soluble $BOD_5$ fraction (USEPA, 1993; WEF, 1998, 2000b). The surface organic removal rate should not exceed 29 g $BOD_5/m^2$ per day on any RBC pass, to limit the loading to within the oxygen transfer capacity of the system.

Hydraulic surface loading rates ($m^3/m^2$ per day, based on the surface area of the medium), applied on organic removal passes, can be evaluated with graphical relationships developed from experimental and full-scale studies. Figure A4 shows a family of curves to approximate the hydraulic loading rate as a function of a range of influent soluble $BOD_5$ concentrations and the required effluent soluble $BOD_5$ at wastewater temperatures higher than 13°C. These curves were based on relations between the effluent soluble $BOD_5$ concentrations and the organic loading rates (kg soluble $BOD_5/m^2$ per day) to the surface of the medium. Predicted performance based on these graphical relationships has been shown to be optimistic, and safety factors based on maximum daily or weekly effluent limitations or peak:average organic load ratio may be used.

The maximum surface ammonium-nitrogen removal rate in the nitrification passes of the RBC system is 1.5 g/$m^2$ per day (USEPA, 1993; WEF, 1998, 2000b). This rate corresponds to a maximum zero-order rate. The maximum nitrification rate occurs in the RBC passes when the soluble $BOD_5$ is less than 15 mg/L, which corresponds to a surface organic loading rate of 3.5 g soluble $BOD_5/m^2$ per day (USEPA, 1993; WEF, 1998, 2000b). At this point, the $cBOD_5$:TKN ratio is low, and the competition between autotrophic and heterotrophic biomass for oxygen is minimized. As the total ammonia-nitrogen concentration is reduced through the nitrification passes of the RBC, the maximum surface ammonia removal rate starts to decrease with ammonium-nitrogen levels less than 5 mg/L (USEPA, 1993; WEF, 2000b). At this point, the rate follows a first-order relationship. Figure A5 shows that the maximum surface total ammonia-nitrogen removal rate decreases as a function of the ammonium-nitrogen concentration.

Hydraulic surface loading rates ($m^3/m^2$ per day) applied to the nitrification passes of the RBC

**Figure A4** **RBC Hydraulic Loading Curves for Soluble $BOD_5$ Removal at $T > 13°C$ (gpd/sf $\times$ 4.074 $\times$ $10^{-2}$ = $m^3/m^2$ per day) (gpd/sf = gallons per day per square foot) (WEF, 1998)**

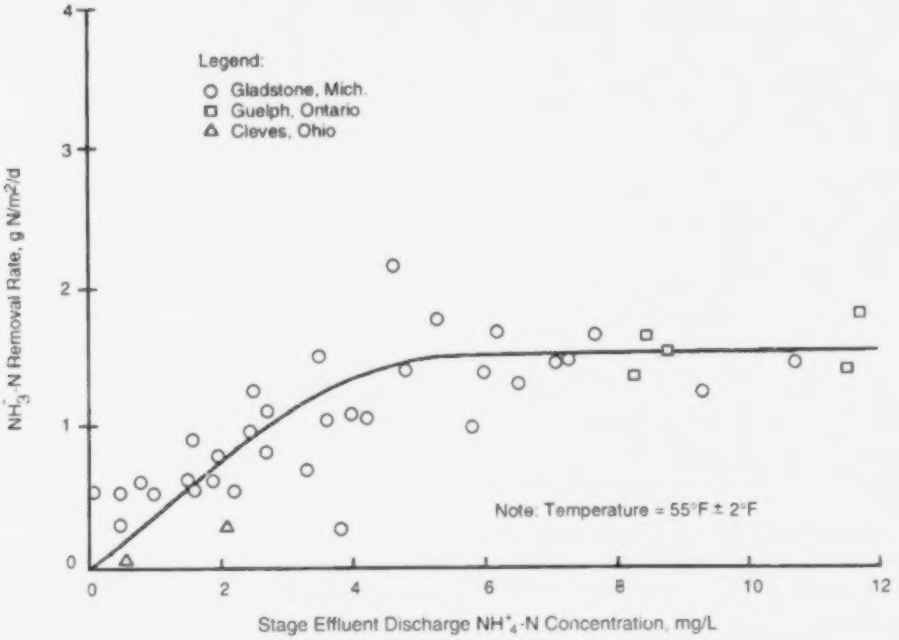


system can be evaluated using graphical relationships. Figure A6 shows a family of curves to approximate hydraulic loading rates as a function of a range of influent total ammonia-nitrogen concentrations and the required effluent total ammonia-nitrogen at wastewater temperatures higher than 13°C. As for Figure A4, the use of a safety factor is recommended.

The hydraulic surface loading rates curves in Figures A4 and A6 are established for temperatures higher than 13°C. Surface organic and ammonia removal rates and microbial growth are influenced by temperature and tend to decrease with lower temperatures. Studies have shown that the maximum zero-order rate of 1.5 g ammonium-nitrogen/$m^2$ per day does not increase with temperatures higher than 13°C but decreases at lower temperatures. Figure A7 provides temperature correction factors for carbonaceous removal and nitrification rates for temperatures less than 13°C. The hydraulic loading rates are then multiplied by the correction factor obtained for a given temperature.

A typical evaluation of the surface area of medium required for influent organic removal is to use the surface removal rates for $BOD_5$ and soluble $BOD_5$. The hydraulic loading rate required to obtain an effluent concentration of 15 mg soluble $BOD_5$/L can be evaluated from Figure A4 and used to calculate the surface area of medium required to achieve this level of treatment. The media surface areas are obtained and

**Figure A5 Maximum Surface Total Ammonia-Nitrogen Removal Rate Decrease as a First-Order Relation with Ammonium-Nitrogen Concentration (USEPA, 1993)**

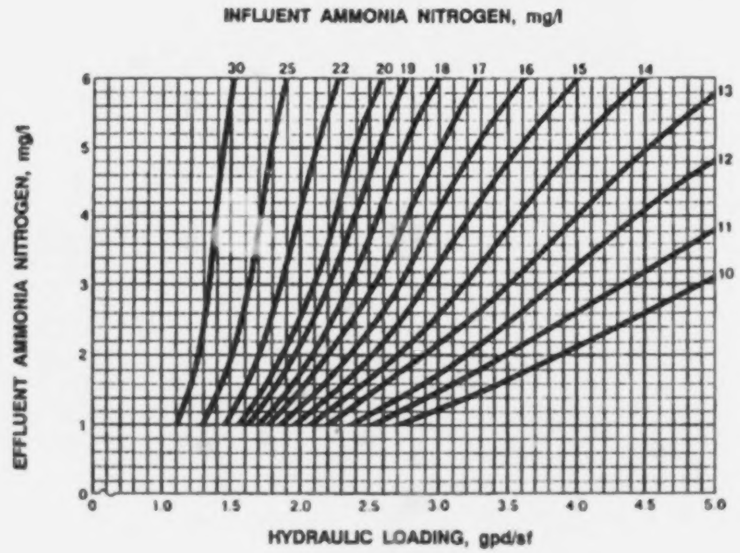


compared, and the most conservative value is used.

Determining the surface area of medium required for the removal of the total ammonia-nitrogen load uses the maximum surface ammonium-nitrogen removal rate, previously mentioned, for a reduction down to a total ammonia-nitrogen concentration of 5 mg/L. Figure A5 can then be used to identify the surface total ammonia-nitrogen removal rate to reduce the total ammonium-nitrogen concentration from 5 mg/L to the effluent total ammonia-nitrogen concentration requirement. The hydraulic loading rate value required to obtain the final effluent total ammonia-nitrogen concentration can be evaluated from Figure A6 and used to evaluate the surface area of medium required to achieve the ammonia removal. The media surface area determinations are then compared, and the most conservative value should be used.

Mathematical models from the U.S. EPA's Watson, Opatken and Benjes may be used to design the RBC surface area requirements for organic load removal and verify the organic removal efficiency of each pass (WEF, 2000b). Mathematical models from Pano may be used to design the RBC surface area requirements for ammonia load removal (WEF, 2000b). The models may use safety factors based on maximum daily or weekly effluent limitations or peak:average organic load ratio and temperature sensitivity relationships.

**Figure A6 RBC Hydraulic Loading Curves for Ammonia Removal at T ≥ 13°C (gpd/sf × $4.074 \times 10^{-2}$ = $m^3/m^2$ per day) (gpd/sf = gallons per day per square foot) (WEF, 1998)**



**Tertiary Nitrification (Two-Stage Process)**

Nitrification in a separate RBC stage is based on the same considerations as for the single-stage process (i.e., combined $BOD_5$ removal and nitrification), and influent soluble $BOD_5$ should be kept lower than 15 mg/L.

The design relationships and the nitrification predictability for single-stage and tertiary nitrification are empirical and not well defined. Process considerations, mentioned hereunder, should be taken into account to ensure satisfactory operation of RBCs.

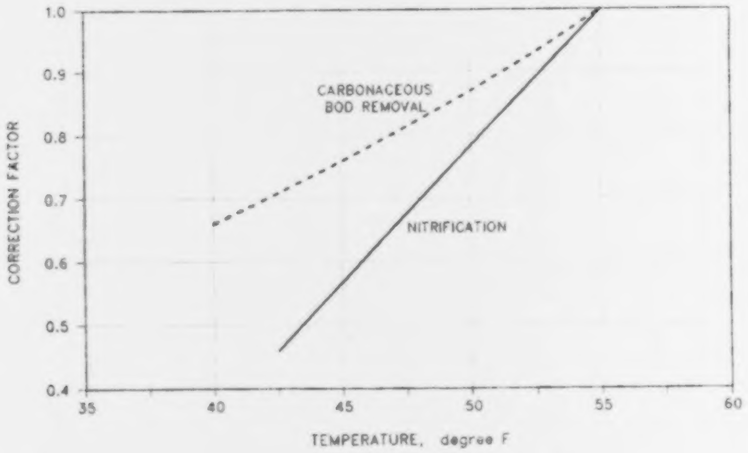
### A.2.2.2 Process Considerations

Some key process considerations are particularly important to provide a sustained low ammonium-nitrogen effluent concentration in RBC systems. This section briefly covers these process considerations.

Cold wastewater temperature negatively affects the rates of organic removal and nitrification in the RBC system. Thus, RBC system efficiencies should be determined for each season and especially for the critical colder periods, where effluent limitations are required.

Alkalinity and pH are two factors of importance for nitrification efficiency in the RBC system, as for other biological treatment processes. Studies have shown that nitrification efficiency is strongly affected by a pH reduction from 7.0 to 6.0. RBC

**Figure A7 RBC Temperature Correction Factors ((°F - 32) × 0.555 = °C) (WEF, 1998)**



systems acclimated to pH levels of less than 7.0 may be able to operate, but in such cases pH has to be maintained within a low degree of variation, and process stability may be a concern. Effluent alkalinity residual in the range of 50–100 mg calcium carbonate/L should be maintained to ensure nitrification process stability and avoid pH drops.

The oxygen requirement for aerobic treatment is provided through intermittent exposure of the medium to the atmosphere. The oxygen transfer rate must be sufficient to ensure aerobic conditions in the active biofilm thickness. The maximum oxygen transfer rate reported for full-scale operating RBCs ranged from 6.8 to 7.3 g oxygen/m$^2$ per day (USEPA, 1993; WEF, 2000b). These oxygen transfer rates correspond to the maximum surface organic and ammonium-nitrogen removal rates previously mentioned. Supplemental compressed air diffusion can be provided.

Sidestream recycle loadings from solid processing unit operations must be taken into consideration in the hydraulic and contaminant loadings applied to the RBC system and in the surface area of medium required. Consideration of these streams and their impacts is important to avoid overloading situations and process efficiency failures.

High hydraulic fluctuations may be detrimental to the RBC process performance efficiency. It has been recommended that flow equalization at a peak:average ratio exceeding 2.5 be incorporated in the RBC system to regulate the applied hydraulic loadings. In the same manner, high and variable organic loadings may have a negative impact on the operation and performance of RBCs. Equalization may reduce influent loading fluctuations and avoid the installation of additional treatment units.

Flow distribution is an important factor to consider in cases of RBC systems with several parallel trains. Flow should be evenly distributed through all the trains to avoid overloading and underloading conditions in the first passes of the trains. Such conditions could lead to severe biofilm thickness variations and weight impacts on shaft operation. Thus, efficient flow distribution between all parallel units must be provided. A low headloss influent channel perpendicular to the parallel trains' flow path and influent weir at the influent front end of the first passes should be installed to ensure proper flow distribution (Figure 5.8). In the case of low flows using one rotating contactor, the shaft should be oriented in the direction of the wastewater. Provision for step feed within the first passes of a train may provide flexibility for more efficient control of the applied influent load.

### A.2.2.3 Advantages/Limitations

RBCs were commercialized in the late 1960s by taking advantage of plastic media development. Severe structural damage to shafts and media from excessive biomass buildup has resulted in the process falling out of favour since the mid-1980s (WEF and ASCE, 1998). The limitations associated with biomass buildup must be understood to successfully use the RBC process. RBC may achieve combined BOD, removal and nitrification and tertiary nitrification. The major advantages related to RBCs are briefly outlined as follows:

- Nitrifying RBCs can produce effluent with total ammonia-nitrogen lower than 5 mg/L.
- Exposure of the biofilm to the atmosphere, from the shaft rotation, provides the process air required for aerobic treatment under normal operation, thus avoiding the use of compressed air.
- Modular process characteristics of the RBC permit the capacity of an existing installation to be easily increased.
- O&M requirements are simple, compared with activated sludge.

In terms of limitations, RBCs have historically been strongly affected by excess biofilm growth. Limitations include the following:

- Space requirements are relatively important due to the horizontal installation of the media arrangement, compared with trickling filters.
- The installation of compressed air may be required in case of overloading to satisfy biomass oxygen requirements, especially in the first passes.
- Excess biofilm growth may clog the void space of the medium, reduce the oxygen transfer efficiency and decrease the substrate and nutrient diffusion into the biofilm. Flow short-circuiting and channelling may then be created, thus reducing the RBC's efficiency.
- Overweight of excess biomass growth has been typically experienced and may provoke structural damage of the shaft and media.
- Uneven influent distribution to the first passes may create overload and underload conditions between trains of RBCs. Overloading may lead to excess biofilm growth and overweight conditions on the shafts.
- Excess biofilm accumulation control is required to avoid process impacts such as overweight on the shaft, media clogging and an inability to maintain rotational speeds. Means to remove excess biofilm growth may include shaft rotational speed control/reversal and air stripping.

- Intermittent exposure of the last pass to high ammonium-nitrogen concentrations may necessitate alternating the feed sequence of last passes to favour the development of a satisfactory nitrifying biofilm in the last pass and maintain nitrification process performance.

Further information on operational problems and possible corrective actions, including overloading and excess biofilm growth, related to RBCs is provided in Section 7.2.2.

SBCs are an alternative to the classic RBCs. SBCs involve the same type of rotating biological contactors, but the submergence is increased between 70 and 90%. The main advantage is to significantly decrease loads on the SBC shaft, thus reducing the risk of structural damage to the shaft. However, the SBC is typically provided with compressed air for shaft entrainment and process requirements. The use of compressed air increases the energy consumption requirements compared with classic RBCs.

### A.2.3 Aerobic Submerged Fixed-Bed Reactors

#### A.2.3.1 Biofor Process

**Process Characteristics**

*Media*

The type of medium used as a support for the biomass and for clarification is an expanded schist, referred to as biolite, with a density varying from 1.4 to 1.8 g/cm$^3$. Two types of biolite are used according to the type of treatment required. Biolite P has a typical diameter of 3.5 mm (3.2–3.8 mm) and is utilized mainly for tertiary nitrification and for dilute wastewater. Biolite L has a typical diameter of 2.7 mm (2.5–2.9 mm) and a specific surface area of 700 m$^2$/m$^3$ (ONDEO Degrémont, technical communication, 2001). It is used mainly for the removal of carbonaceous matter at high filtration rates and has been applied in tertiary nitrification mode (ONDEO Degrémont, technical communication, 2001). The medium depth in the Biofor process varies from 2.9 to 3.9 m according to the type and level of treatment desired and the corresponding biomass requirement.

*Tertiary Nitrification (Two-Stage Systems)*

Influent wastewater with high organic concentrations may be nitrified in a two-stage system. The nitrification process is sensitive to the cBOD$_5$:TKN ratio, oxygen availability, total ammonia-nitrogen concentration, temperature, pH and alkalinity. In a two-stage system, the total ammonia-nitrogen loading, the effluent quality requirement and temperature effects characterize the tertiary nitrification process.

A study by Pujol *et al.* (1993) identified limiting ammonium-nitrogen loading rates of 1.5 kg/m$^3$ per day and 10 m$^3$/m$^2$ per hour for the nitrification process (WEF, 2000b). The ammonium-nitrogen loading rate is based on the volume of medium, and the hydraulic loading rate is based on the plan area of the filter. Nitrification tests conducted for the Acheres, Paris, wastewater treatment plant secondary effluent (Vedry *et al.*, 1994) have shown the Biofor process to remove 0.85 kg ammonium-nitrogen/m$^3$ per day with a hydraulic rate of 7 m$^3$/m$^2$ per hour at a temperature of 14°C (WEF, 2000b).

A pilot study of three-stage biological filters for total nitrogen removal (Aridgides *et al.*, 2000) for the Binghamton–Johnson City treatment plant, New York, has shown the tertiary nitrification Biofor unit to produce at average conditions an effluent of 3.0 mg ammonium/L at a loading rate of 0.3 kg ammonium-nitrogen/m$^3$ per day and a hydraulic rate of 4.6 m/h. The same study indicated, at maximum conditions, an effluent of 5.0 mg ammonium-nitrogen/L at a loading rate of 0.51 kg ammonium-nitrogen/m$^3$ per day and a hydraulic rate of 7.6 m/h. The removal efficiencies at maximum conditions were influenced by high $cBOD_5$:TKN ratios.

Volumetric loading rates (kg/$m^3$ per day) are set according to the effluent requirement and are used to determine the required media volume. The hydraulic loading rate ($m^3/m^2$ per hour) determines the cycle time between backwash events. The surface of the filter and the media height are evaluated according to this parameter.

*Process Considerations*

Some key process considerations are particularly important to provide a sustained low total ammonia-nitrogen effluent concentration with the Biofor process.

Alkalinity in the wastewater applied to the Biofor reactor should be sufficiently high to ensure buffering capacity and to maintain an adequate residual in the treated effluent. This measure should help to prevent pH variations and associated effects on nitrification activity. Low-temperature effects must be taken into consideration with loading rate evaluations.

Fine screening of the raw wastewater is considered as essential, since the upward-flowing water comes into contact with the injection nozzles before the biological and the filtration activities in the Biofor process. Thus, any possibility of nozzle blockage has to be prevented with the use of 2.5-mm fine mechanical screening before the wastewater enters the Biofor process (ONDEO Degrémont, technical communication, 2001).

Backwashing of the filter medium is an important feature in the operation of the Biofor process. This activity enhances the control of excess biomass developed on the medium and promotes cleaning the medium to maintain its filtration capacity. The backwash has to be performed on a regular basis. At a preset headloss value, an automatic timer or the quality of the treated water may control backwashing frequency. The preset headloss value usually ranges between 1.50 and 2 m, according to the biolite type in use. The typical backwashing frequency reported for the Biofor process with carbonaceous removal is 24 hours and for Biofor tertiary nitrification process is 48 hours (ONDEO Degrémont, technical communication, 2001). The overall duration of a backwash event is approximately 40 minutes.

Medium loss in the treated effluent is controlled and prevented using a medium trap. However, some medium is lost during backwashing due to high velocities. The loss is estimated to be less than 3% per year (ONDEO Degrémont, technical communication, 2001). Medium attrition should be monitored to ensure long-term integrity of the grains.

Recycle loads from solids processing returned ahead of primary treatment should be taken into consideration in the hydraulic and organic loadings applied to the Biofor process to achieve and maintain the ammonia removal requirement.

### A.2.3.2 Biostyr Process

#### Process Characteristics

*Media*

The medium used in the Biostyr process is a floating expanded polystyrene spherical bead, referred to as biostyrene. The medium sizes range from 3 to 6 mm, and the density of the biostyrene is reported as 0.04 g/$cm^3$ (John Meunier Inc., technical communication, 2001). The choice of medium size depends on the effluent quality requirement for total ammonia-nitrogen, the level of effluent suspended solids, the hydraulic rate ($m^3/m^2$ per hour) and the hydraulic peaking factor. Reports have shown that biostyrene in the high size range was tested in case of TKN and COD loads higher than design parameters and has been shown to provide a reduction of the backwashing frequency. The reported specific surface area for the biostyrene with a nominal diameter of 3.6 mm is 1200 $m^2/m^3$ (Heist *et al.*, 1999); thus, a high surface area for the development of a fixed-film biological growth is available. The medium depth in the Biostyr process may vary from 2.5 to 4.0 m and depends on the type and level of treatment desired and the biomass required (John Meunier Inc., technical communication, 2001).

### *Tertiary Nitrification (Two-Stage Systems)*

Tertiary nitrification with a two-stage system involves a secondary-treated effluent with low $BOD_5$ and high total ammonia-nitrogen content. This secondary effluent may originate from a previous Biostyr reactor or any other biological treatment. If the $BOD_5$ concentration of Biostyr process influent is significant, it becomes oxidized first in the filter biomass, and the nitrification process occurs farther up within the filter bed where BOD is significantly lower. Tertiary nitrification process efficiency depends on the cBOD:TKN ratio, oxygen availability, total ammonia-nitrogen concentration, temperature conditions, pH and alkalinity.

Nitrification tests conducted for the Acheres, Paris, wastewater treatment plant secondary effluent (Vedry *et al.*, 1994) have shown the Biostyr process to remove 1.67 kg ammonium-nitrogen/$m^3$ per day from an influent load of 1.87 kg ammonium-nitrogen/$m^3$ per day, a percentage removal of 89% (WEF, 2000b). Further studies and follow-up (Payraudeau *et al.*, 1999) have shown that nitrification activity is reduced at temperatures lower than 14°C and applied loads greater than 1.5 kg TKN/$m^3$ per day. The ammonium-nitrogen and TKN loading rate are based on the volume of the medium.

A pilot-scale study in Syracuse, New York (Heist *et al.*, 1999), has shown that at a wastewater temperature of 13–14°C, an average applied ammonium-nitrogen load of 1.2 kg/$m^3$ per day at a hydraulic rate of 10 m/h was able to produce a treated water with less than 1 mg ammonium-nitrogen/L on a daily composite sample. The pilot was conducted with biostyrene media in the high size range. The hydraulic loading rate was based on the plan area of the filter.

Pilot tests and full-scale installation at the Davyhulme wastewater treatment plant in Manchester, U.K. (Wicquart *et al.*, 1999), have shown the Biostyr process to reach effluent concentrations of less than 1 mg ammonium-nitrogen/L at an average applied load of 0.42 kg ammonium-nitrogen/$m^3$ per day, with an optimum hydraulic rate of 8 m/h at nominal flow. At a peak load of 0.94 kg ammonium-nitrogen/$m^3$ per day, the effluent ammonium-nitrogen rose but stayed under 5 mg/L. The lowest temperature recorded was 7°C. The Biostyr process used 3.5-mm biostyrene medium.

The volumetric loading rate (kg/$m^3$ per day) is used to determine the medium volume required, while the hydraulic loading rate ($m^3/m^2$ per hour) permits the determination of filter surface area and medium height.

### *Process Considerations*

Process considerations of particular importance to provide a sustained low total ammonia-nitrogen effluent concentration with the operation of the Biostyr process are similar to other aerobic submerged fixed-bed reactors in terms of alkalinity, pH, temperature and recycle loads.

Medium screening of the raw wastewater is required to avoid rapid headloss buildup in the filter and high frequency of backwashing. Thus, the use of 10-mm medium mechanical screening to remove a fraction of the solids from the wastewater before it enters the Biostyr process is suggested.

The backwashing activity is required for the control of the excess biomass developed on the surface of the medium and the removal of suspended solids retained in the medium from the filtration process. The backwash events are performed on a regular basis, and the frequency is controlled by a preset headloss value, an automatic set time limit or the quality of the treated water. The typical backwashing frequency reported for the Biostyr process is 24 hours (John Meunier Inc., technical communication, 2001). The overall duration of a backwash event is approximately 20 minutes.

It is reported that medium loss does not occur during any of the treatment phases. However, potential medium loss during the backwashing event due to high velocities should be verified and addressed, if required. Medium attrition should be paid attention to, to ensure long-term integrity of the grains.

### *Advantages/Limitations*

Aerobic submerged fixed-bed reactors (BAF) may achieve tertiary nitrification and can produce effluent with total ammonia-nitrogen lower than 5 mg/L. In the case of the single-stage process with combined carbon removal and nitrification, the nitrification efficiency may be lower than for tertiary nitrification and should be addressed in accordance with the effluent total ammonia-nitrogen concentration requirement, and pilot trials should be conducted. The major advantages related to aerobic submerged fixed-bed reactors are outlined as follows:

- Biological treatment and suspended solids filtration are achieved simultaneously.
- Volumetric organic loading rates are higher than classic biotowers.
- Secondary clarification is not required.
- Modular and compact process characteristics permit easy upgrading of the capacity of an existing installation.
- Site requirements and footprint of the reactors are low.
- Modular biofilters (BAF) installed in series can be switched from series to parallel treatment when high hydraulic flows occur to permit treatment and filtration of the total hydraulic load.

In terms of limitations, the following points may be outlined:

- Excess biofilm growth on the medium and the retained suspended solids must be evacuated on a periodic basis. Sloughing of excess biomass and the trapped solids is achieved through backwashing cycles. Close monitoring of the frequency of the backwashing cycles must be performed to detect overloading of the filters or backwashing deficiencies.
- Medium loss with the treated effluent or the backwash water may occur, and replacement of the medium must be evaluated.
- Medium attrition potential should be verified to ensure long-term integrity of the grains.
- Treated effluent recirculation may be required in some cases to maintain a minimum or favour a constant filtration velocity through the medium.

## A.2.4 Aerobic Submerged Mobile-Bed Reactors

### A.2.4.1 Fluidized Bed Reactors

#### Process Characteristics

##### *Media*

The type of medium used for biofilm support is sand clay with an average dimension of 0.3 mm. The medium has a density of 1.55 $g/cm^3$ and a specific surface area of 9000 $m^2/m^3$ (John Meunier Inc., technical communication, 2001).

##### *Tertiary Nitrification (Two-Stage Systems)*

The tertiary nitrification process and applied loads depend on and vary with the influent characteristics and physical parameters. The ammonium-nitrogen volumetric load reported to be applied on the reactor at 10°C is 0.62 $kg/m^3$ per day (John Meunier Inc., technical communication, 2001). The volumetric loading rate is based on the reactor volume.

##### *Process Considerations*

Process considerations include parameters such as temperature, pH and alkalinity. Recycle loads from solids processing should be taken into consideration in the loads applied to the fluidized-bed nitrification process to achieve and maintain the ammonia removal requirement.

Medium loss in the effluent is estimated to be 15–20 $g/m^3$ of treated water (John Meunier Inc., technical communication, 2001). As with fixed-bed reactors, medium attrition should be a factor to monitor to ensure long-term integrity of the grains.

Medium bed expansion quality should be characterized to ensure that the specific surface area is available for biofilm growth. The Biolift fluidized-bed process comprises water traps at various levels, which permits the installation of pressure sensors. The headloss measure at each level depends on the sand concentration.

##### *Advantages/Limitations*

Fluidized-bed reactors have historically been applied in industrial wastewater facilities with limited application in municipal wastewater treatment (e.g., Biolift process). The fluidized-bed reactors may achieve combined $BOD_5$ removal and nitrification, as well as tertiary nitrification. The fluidized medium offers a high specific surface area, which permits total ammonia-nitrogen loading rates higher than those with traditional attached-growth technologies.

In terms of limitations, the following points are outlined:

- Operation of fluidized-bed reactors has shown problems with the control of bed expansion and biofilm thickness, influent distribution and oxygen saturation.
- Medium bed expansion should be closely characterized to ensure availability of the medium surface area for biofilm growth.
- Excess biofilm growth is achieved either in a separate sand-cleaning system or within a reactor internal recirculation device, such as an airlift (e.g., Biolift reactor), to provide the required turbulence to slough off excess biomass.
- Medium loss with the treated effluent may occur, and replacement of the medium must be evaluated.
- Medium attrition potential should be verified to ensure its long-term integrity.
- O&M requirements are higher than for traditional attached-growth technologies.
- Operational problems and corrective actions are to be completed with existing and future installations of fluidized beds in MWWTPs.

### A.2.4.2 Moving-Bed Biofilm Reactor

**Process Characteristics**

*Media*

The Kaldnes MBBR medium consists of an extruded high-density polyethylene small cylinder with an inside crosspiece and outside longitudinal fins. The medium has a specific density of 0.95 g/cm$^3$ (Kaldnes, technical communication). Two types of media, K1 and K2, are currently used for municipal wastewater treatment applications. Medium type K1 has a diameter of 10 mm, a height of 7 mm and a specific surface area of 500 m$^2$/m$^3$ (Kaldnes, technical communication). Medium type K2 has a 15-mm diameter and a 15-mm height with an inner diameter of 5 mm and a series of separation walls between the outer and inner rings. The K2 medium has a specific surface area of 350 m$^2$/m$^3$ (WERF, 2000).

The media types are designed for different treatment applications. The K1 type is used in most of the municipal and in all industrial effluent treatment applications. The K2 type is used in municipal wastewater treatment applications where primary treatment comprises only coarse screens without primary clarification. The fill percentage of medium in the empty reactor varies according to the volumetric and hydraulic loadings, the degree of treatment required, influent characteristics including temperature and the oxygen transfer capability. The maximum fill capacity is approximately 70% v/v (WERF, 2000).

*Combined BOD$_5$ Removal and Nitrification (Single Stage)*

Influent wastewater circulation downstream of the MBBRs leads to the development of a biofilm as a function of the available substrate (i.e., food). Thus, the biomass in the first reactor is composed mainly of heterotrophic organisms associated with cBOD$_5$ removal. As the cBOD$_5$ load is reduced, the attached-growth biomass in the second or following reactors comprises a higher degree of nitrifiers. Reduction of influent BOD$_5$ loading and the cBOD$_5$:TKN ratio to a low level reduces the heterotrophic organisms, and oxygen will be available for the nitrifiers to perform nitrification.

As for other attached-growth systems, nitrification in the MBBR process depends on organic and total ammonia-nitrogen loads, cBOD$_5$:TKN ratio, level of total and soluble influent BOD$_5$, oxygen availability, total ammonia-nitrogen concentration and temperature conditions. The rate of nitrification is reported to depend on DO and temperature. The rate of nitrification is influenced by hydraulic, organic and total ammonia-nitrogen loading rates.

In a single-stage application, the total surface area of the medium required to achieve complete treatment is the sum of the surface medium for degradation of the organic loading and for nitrification.

The biofilm surface loading rate in the first reactor required to remove BOD to favour nitrification activity in the following reactors is reported as 5.2 g BOD$_5$/m$^2$ per day at a temperature of 15°C (WERF, 2000). This surface loading rate assumes a DO concentration greater than 3 mg/L. The surface loading rate is based on the surface of the medium.

The ammonium-nitrogen surface loading rates reported for nitrification are 1.0 g/m$^2$ per day with an ammonium-nitrogen concentration higher than 3 mg/L and 0.45 g/m$^2$ per day with an ammonium-nitrogen concentration lower than 3 mg/L (WERF, 2000). These surface loading rates assume a DO concentration greater than 5 mg/L and a temperature of 15°C. The biodegradable COD concentration should be assumed to be less than 20 mg/L. Other studies reported surface loading rates of 1 g ammonium-nitrogen/m² per day at 10–13°C from a bench-scale MBBR system and 0.95 g ammonium-nitrogen/m² per day at 13°C at the Lillehammer MBBR plant in Norway (WERF, 2000).

The medium and reactor volumes required are calculated according to the specific surface area (m²/m³) of the medium type used and the medium fill percentage in the reactors. Temperature sensitivity relationships may be used for surface loading rate corrections.

### *Tertiary Nitrification (Two-Stage Process)*

Nitrification in a separate MBBR stage is based on the same considerations as for the single-stage nitrification process. Influent soluble COD should be kept lower than 20 mg/L.

### *Process Considerations*

Process parameters to be taken into consideration include factors such as temperature, pH, alkalinity buffering capacity and recycle loads from solids processing.

Medium retention sieves or screenings should have 5-mm openings for the K1 medium and 12-mm openings for the K2 medium. An aeration pipe under the sieve assembly should be installed to ensure continuous circulation of the media in the vicinity of the sieve. This device minimizes clogging of the sieve by the media and also cleans the medium retention screen.

In multiple reactors in series with nitrification activity in the last reactors, the last reactor may be intermittently exposed to high total ammonia-nitrogen concentrations. Maintenance of nitrification activity in the last reactor may be obtained by alternating the sequence of the last two reactors on a weekly basis. This favours the development of a better nitrifying biofilm in the last reactors.

### *Advantages/Limitations*

MBBRs have been operated in municipal treatment applications mostly located in northern Europe and may achieve combined $BOD_5$ removal and nitrification and tertiary nitrification. Advantages related to MBBRs are outlined as follows:

- MBBRs can produce effluent with a total ammonia-nitrogen concentration lower than 5 mg/L.
- Existing or unused tankage can be retrofitted for the installation of the MBBR process.
- Land area for MBBR is typically smaller than for activated-sludge treatment.
- MBBR media can be added in existing activated sludge tanks to increase the biomass content, organic oxidation rates and nitrification capacity.

In terms of limitations, the following points are outlined:

- Excess biofilm growth on the medium is sloughed off by the mixing turbulence created by the coarse-bubble diffusers. Turbulence to control excess growth should be closely monitored to avoid biomass accumulation problems on the medium.
- Sieve assemblies at the front end of each MBBR cell are required to retain the free-moving medium within the cells.
- Potential clogging of the sieves may be minimized by the use of an air pipe under the sieve assemblies. The air flow rate ensures circulation of the medium near the sieve and cleaning of the sieve.
- Intermittent exposure of the last MBBR cell to high ammonium-nitrogen concentrations may necessitate alternating the feed sequence of the last cells to favour the development of a satisfactory nitrifying biofilm in the last cell and maintain nitrification process performance.

## A.2.5 Recirculating Sand Filter

### A.2.5.1 Process Characteristics

*Tertiary Nitrification (Two Stage)*

Recirculating sand filtration technology represents a relatively low-cost upgrade option for tertiary nitrification of lagoon process effluents. As for other attached-growth systems, the nitrification process in the recirculating sand filter is sensitive to nutrient diffusion efficiency in the layer of biofilm covering the medium, cBOD$_5$:TKN ratio, level of total and soluble influent BOD$_5$, oxygen availability, total ammonia-nitrogen concentration, temperature, pH and alkalinity. In the case of effluent lagoon tertiary treatment, the BOD$_5$ and solids concentration in the lagoon effluent should typically be less than 20 mg/L. Temperature variations in cold climates are a concern, as nitrification rates are affected by low temperature.

The recirculating sand filter has been shown to achieve more than 80% nitrification efficiency of the applied total ammonia-nitrogen load under aerobic conditions within the filter bed (Ministère de l'Environnement du Québec, 1992).

Recirculating sand filters are typically operated with hydraulic loadings ranging from 0.12 to 0.20 $m^3/m^2$ per day (excluding the recirculated flow), effluent recirculation ratios of 3:1 to 5:1 and an applied wastewater dosing frequency of one dosage per 30 minutes (Metcalf & Eddy, 1991; Ministère de l'Environnement du Québec, 1992, 2001). Recirculation rates may vary according to the level of treatment required and the influent characteristics.

A pilot field study at Jaffrey, New Hampshire (Rushbrook and Urban, 1998), has shown that tertiary nitrification of an aerated lagoon effluent during winter conditions can reduce total ammonia-nitrogen by up to 85%, with discharge concentrations of between 0.1 and 4.4 mg/L. The recirculating sand filter was operated with a hydraulic loading rate of 0.08–0.11 $m^3/m^2$ per day, a recirculation ratio of 3:1 and lagoon effluent temperatures as low as 3°C. The recirculating sand filter was operated during the winter period of November to April.

A full-scale application for treating the effluent of a small-flow aerated lagoon with continuous discharge has been conducted at Stetsonville, Wisconsin (Olson and Danen, 2001). The recirculating sand filter for tertiary treatment has been operating since September 2000 and has gone through a complete season of wintertime operation in 2000–2001. The results indicate that the average recirculating sand filter discharge total ammonia-nitrogen concentration was 1 mg/L, with 90% removal efficiency, during the winter period, with effluent temperature recorded between 5 and 6°C over a two-month period.

### A.2.5.2 Process Considerations

Process considerations to provide a sustained low total ammonia-nitrogen effluent concentration include similar factors as have been identified for other technologies in terms of temperature, pH and alkalinity.

Overloading of recirculating sand filters should be prevented to avoid saturation of the attached-growth surface layer of the granular filter bed, which may lead to total ammonia-nitrogen breakthrough in recirculating sand filter discharge before the completion of nitrification. The pressurized distribution system should permit a uniform application of the applied wastewater on the entire filter bed area. In the case of saturation of the recirculating sand filter surface through solids buildup, a rest period of the cell may permit regeneration of the surface of the medium, or else the saturated layer of the medium may be manually removed.

The point of discharge of the recirculated treated effluent in the recirculation/dilution tank should be located in the vicinity of the secondary effluent inlet to favour oxygenation of the applied wastewater to the filter bed.

Cold-climate application of the recirculating sand filtration technology may include some of the following considerations to favour nitrification efficiencies:

- insulated cover over the filter to reduce heat loss to the ambient air;
- insulation of the recirculation/dilution tank to reduce heat losses to surrounding soil;
- drainage of the water surface from the treatment area to reduce freezing potential of the filter surface and avoid anoxic/anaerobic conditions;
- decrease the treated effluent recirculation ratio during low-temperature periods to minimize applied wastewater temperature reduction;
- complete or partial flushing of the pressurized distribution system between wastewater application cycles to avoid freezing;
- variable effluent lagoon discharge points to ensure sufficient ammonia load to the recirculating sand filter, as lagoon efficiency varies according to season, temperature, HRT and ice cover on the lagoon; and
- recirculating sand filter offers potential application for cold weather year-round effluent quality requirements related to ammonia removal, based on pilot field study (Rushbrook and Urban, 1998).

### A.2.5.3 Advantages/Limitations

Recirculating sand filters possess the advantage of achieving tertiary nitrification of small-flow lagoon effluent with continuous discharge (year-round) to below 5 mg/L of total ammonia-nitrogen. Recirculation of the treated effluent enhances oxygenation of the wastewater prior to its application onto the filter bed. Recirculating sand filtration represents a relatively low cost option for tertiary nitrification of lagoon process effluent in cases where land is available and its capital cost is high, even if O&M requirements of the recirculating sand filter are higher than for intermittent sand filtration and constructed wetlands. Recirculating sand filter technology offers additional beneficial features in comparison with the intermittent sand filter:

- treated effluent recirculation ratio flexibility to adjust applied hydraulic load and contaminant concentrations during seasonal variations;
- total ammonia-nitrogen load passes several times through filter bed medium, which enhances ammonia oxidation efficiency;
- reduced filter bed surface area requirement due to higher hydraulic load rate; and
- low potential odour problems and clogging due to diluted load.

In terms of limitations, recirculating sand filters may experience solids buildup and saturation of the granular filter bed and isolation for regeneration of the medium, or manual removal of the saturated layer may be required.

### *A.2.6 Intermittent Sand Filter*

#### A.2.6.1 Process Characteristics: Tertiary Nitrification (Two Stage)

As for other attached-growth systems, nitrification in the intermittent sand filter process is sensitive to nutrient diffusion efficiency in the layer of biofilm covering the medium, cBOD$_5$:TKN ratio, level of total and soluble influent BOD$_5$, oxygen availability, total ammonia-nitrogen concentration, temperature, pH and alkalinity. In the case of lagoon effluent tertiary treatment, the BOD$_5$ and solids concentration in the lagoon effluent should be less than 20 mg/L. Temperature variations in cold climates are a major concern with intermittent sand filtration, as tertiary nitrification has been shown to be strongly affected by low wintertime temperature (McMaster University and WTI, 1996; Rushbrook and Urban, 1998).

Intermittent sand filters are typically operated with hydraulic loading of 0.04 $m^3/m^2$ per day and a dosing frequency per filter bed of 2–6 times per day with an optimum at 4 times per day (Metcalf & Eddy, 1991; Ministère de l'Environnement du Québec, 1992, 2001). The equilibrium between the hydraulic loading rate and the dosing frequency permits the optimized distribution of wastewater over the filter bed surface area for enhanced treatment efficiency.

Two wastewater treatment facilities at New Hamburg and Schomberg, Ontario, have used intermittent sand filtration for polishing of effluent from aerated and facultative lagoons, including the removal of total ammonia-nitrogen (OMOE and Environment Canada, 1993; McMaster University and WTI, 1996). The intermittent sand filtration processes were operated from spring to late autumn at New Hamburg and only during spring and fall at Schomberg. During periods of non-discharge, the wastewater is retained in storage lagoons. Thus, year-round operation of the intermittent sand filter is not conducted. Sampling at New Hamburg during the first days of operation after the winter storage period has shown the intermittent sand filter to immediately initiate nitrification at a wastewater temperature of 4°C. Intermittent sand filtration continuous winter operation was attempted at New Hamburg, but was unsuccessful due to freezing problems.

Winter operation of intermittent sand filters is reported in the literature but with a reduction of the effluent quality during this period due to the effect of decreasing temperature on biological treatment (McMaster University and WTI, 1996). A pilot field study at Jaffrey, New Hampshire (Rushbrook and Urban, 1998), has shown that tertiary nitrification of an aerated lagoon effluent during winter conditions with an intermittent sand filter, although achieving a good-quality effluent, could not meet the total ammonia-nitrogen limit of 5.6 mg/L during this period compared with more successful recirculating sand filter trials under equivalent treatment conditions.

### A.2.6.2 Process Considerations

Process considerations to provide a sustained low total ammonia-nitrogen effluent concentration during seasonal lagoon discharge include similar factors identified for other technologies in terms of pH and alkalinity buffering capacities.

Overloading of intermittent sand filters should be prevented to avoid saturation of the attached-growth surface layer of the granular filter bed, which may lead to total ammonia-nitrogen breakthrough in intermittent sand filter discharges before nitrification is complete. In the case of saturation of the intermittent sand filter surface through solids buildup, a rest or idle period of the filter bed may permit regeneration of the surface of the medium.

### A.2.6.3 Advantages/Limitations

Intermittent sand filters may achieve tertiary nitrification of small-flow lagoon effluent to below 5 mg/L of total ammonia-nitrogen, but only with seasonal discharge. Low temperature during the winter period strongly affects the nitrification efficiency of the intermittent sand filter technology, thus limiting its application to seasonal treatment. Winter storage capacity of wastewater should then be provided. Intermittent sand filtration represents a low-cost option for tertiary nitrification of lagoon process effluent with seasonal discharge according to land capital cost level, even if its O&M costs are higher than for constructed wetlands.

In terms of other limitations, intermittent sand filtration requires higher land surface area compared with recirculating sand filtration, as the hydraulic loads applied are lower than for the recirculating sand filtration process. Intermittent sand filters may experience solids buildup and saturation of the granular filter bed, and isolation for regeneration of the medium may be required.

## A.2.7 Constructed Wetlands

### A.2.7.1 Process Characteristics: Tertiary Nitrification (Second Stage)

Nitrification in constructed wetlands is sensitive to HRT, cBOD$_5$:TKN ratio, total and soluble influent BOD$_5$, oxygen availability, total ammonia-nitrogen concentration, temperature, pH and alkalinity. In the case of lagoon effluent tertiary treatment, the BOD$_5$ and solids concentration in the lagoon effluent should be less than 20 mg/L.

Studies report that a lagoon effluent residual BOD$_5$ lower than 15 mg/L must be achieved to favour nitrification activity and avoid oxygen competition with heterotrophic organisms in the downstream wetland tertiary treatment stage. The presence of higher BOD$_5$ concentration in the influent to the wetland can delay the nitrification process, and additional HRT will be required to achieve the same level of total ammonia-nitrogen removal.

The influent total ammonia-nitrogen load for design of the constructed wetland should be based on the assumption that all of the TKN load is completely converted to total ammonia-nitrogen to ensure that sufficient HRT in the wetland is provided. The organic nitrogen portion of the TKN is present in the suspended solids and algal cells entering the wetland. Ammonification of the organic nitrogen increases the total ammonia-nitrogen load on the wetland and should then be considered in the design conditions.

A pilot study at Listowel, Ontario, has shown that an FWS wetland can reduce the total ammonia-nitrogen concentration of a facultative lagoon effluent to 4 mg nitrogen/L (NovaTec Consultants Inc., 1996). A full-scale constructed wetland treatment system has the potential to nitrify and reduce the total ammonia-nitrogen content of a lagoon effluent to achieve a final concentration of 5 mg/L.

Temperature variations in cold-climate applications are a concern, as nitrification rates decrease with low temperature. Low temperature may lead to ice cover formation in FWS wetlands, which may lead to atmospheric oxygen transfer reduction, wastewater flow constriction under the ice cover and short-circuiting and, finally, nitrification failure. For the VSB, oxygen may still be available at the roots and rhizomes during the winter months from atmospheric oxygen transfer occurring through the stem passages of dead aquatic plants. Thermal analysis may help to ensure that the constructed wetlands will be physically stable and to permit nitrification reactions to occur during the winter period.

Evaluation of the biological total ammonia-nitrogen removal efficiency and the calculation of the required wetland size can be approximated with iterative equations and models found in the literature. The same procedure is used for the FWS wetlands and the VSB; only the constants and the model types are different. The water temperature in the wetlands during a given period is approximated, and the surface area of the wetland required to achieve the treatment during the coldest month of the year is calculated with ammonia removal models at a given HRT.

Heat transfer models are then used with the previous surface area to calculate the theoretical water temperature in the wetland. The temperature obtained is compared with the initial temperature, and the calculation is repeated until the temperatures converge at a given surface area. In the case of FWS wetlands, heat transfer models are applied to estimate the ice cover thickness over a given period and permit the operating water depth in the winter months to be determined.

Once the required wetland surface area is calculated, hydraulic models are applied to evaluate the length and width configuration compatible with a selected hydraulic gradient to maintain the desired flow conditions in the wetland system.

### A.2.7.2 Process Considerations

Process considerations to provide a sustained low total ammonia-nitrogen effluent concentration include factors such as temperature, pH and alkalinity buffering capacities. The approach to configuring constructed wetlands may include some of the following issues and considerations:

- The total wetland surface area should be divided into a number of smaller parallel cells, as one big cell only may lead to wastewater flow short-circuiting and maintenance difficulties.
- Two parallel cells are recommended as a minimum for the VSB, and three parallel cells are reported to alternate the wastewater flow passage on a frequent basis to favour the growth of root systems of aquatic plants and allow atmospheric reaeration of the medium.
- Parallel cells may be further divided into subcells in series to optimize hydraulic control.
- Slight bottom slope in the water flow direction may be provided to facilitate drainage requirements.
- An inlet distribution system to provide uniform flow and load conditions may involve weir structures or perforated manifold pipes on the total width of the wetland cell and located above the surface with thermal protection or on the bottom of the cell within coarse media.
- An outlet discharge system may involve weir structures or perforated pipes at the bottom of the cell that could be located within coarse media.
- The outlet structure should permit water level adjustment and drainage of the up-front wetland cells.

Aquatic vegetation should be established to an appropriate density to favour rapid maturity of the constructed wetlands. Aquatic plants can be developed in the wetlands either from seeds, which may require a significant period of time, or from rhizome transplants.

The development and penetration of the root systems of aquatic plants in the medium depth of the VSB may be enhanced by gradual water level reduction in the fall period and rotation of filling of parallel cells to favour nutrient consumption and deeper root penetration in the standby cell.

Harvesting of the aquatic vegetation is not usually required, but the dead vegetation in the FWS wetlands can be removed to avoid wastewater flow short-circuiting.

Cold-climate application of the constructed wetlands technology can include some of the following considerations:

- Both constructed wetlands, FWS wetlands and VSBs, with water temperatures lower than 1°C for prolonged periods may lead to system failure; winter storage of the wastewater and seasonal wetland treatment may be required.
- Surface water level elevation of the FWS wetlands in the fall and winter periods may permit increases of the HRT, enhance the biological removal of ammonia during cold

periods, albeit at lower biological rates, and ensure protection against freezing.

- Important surface area requirements and related high HRT for the FWS wetland during extended periods with low air temperatures may increase the risk of freezing of the wetland; thus, the FWS wetland may not be a year-round treatment system under such conditions, as important ice depth formation may lead to physical system failure.
- Variable effluent lagoon discharge points are needed to ensure sufficient total ammonia-nitrogen load to the constructed wetlands, as lagoon efficiency varies according to season, temperature, HRT and ice cover on the lagoon.

### A.2.7.3 Advantages/Limitations

Constructed wetlands can be considered as an upgrading option for tertiary nitrification requirement of small-flow lagoon effluents on a continuous or seasonal basis according to climate conditions. Constructed wetlands represent a low-cost option for tertiary nitrification of lagoon process effluent in cases where land is available and its capital cost is low. O&M requirements for constructed wetlands are typically lower than for recirculating and intermittent sand filters.

Pressurized compressed air distribution headers could be installed under the influent distribution structures of the wetlands and the treatment zones to enhance oxygen availability for both FWS wetlands and VSBs.

VSBs possess the advantages of low odour and reduced mosquito generation and can be contemplated for locations close to habitations. VSBs offer higher ammonia removal rates than FSW wetlands, resulting in wetlands with smaller surface area.

In terms of limitations, constructed wetlands are dependent on site-specific factors, including relief, bedrock proximity, soil permeability, seasonal groundwater table depth and flooding potential. Depending on wintertime temperature conditions, especially when temperatures are lower than 1°C, a severe reduction or failure of the nitrification process may occur with FWS wetlands and VSBs. In such cases, seasonal wetland treatment may be contemplated along with the winter storage capacity of wastewater in the lagoons. Biological and physical stability of the constructed wetlands in the winter period can be approximated with thermal analysis.

Other limitations with FWS wetlands include the following:

- Ammonia removal rates of FWS wetlands are lower than those with VSBs, which results in higher HRT and surface area requirements compared with VSBs.
- High HRT and surface area requirements increase the freezing potential of FWS wetlands.
- Low temperatures in the winter may lead to ice cover formation, operating water depth reduction, oxygen transfer reduction from the atmosphere and wastewater flow short-circuiting.
- Surface water level may need to be increased in the winter period to ensure biological treatment and protection against low temperatures.
- FWS wetlands may be the more economical system to build at design flow rates greater than 400 $m^3/d$, if suitable land is available at reasonable cost (NovaTec Consultants Inc., 1996).

Other limitations with VSBs include the following:

- Aquatic plants with deepest root penetration should be favoured.
- Root system penetration should be enhanced with wastewater flow alternating between parallel VSB cells and surface water reduction in the fall period.
- VSBs are used with light organic load only for nitrification, such as lagoon effluent, considering the restricted oxygen capacity of VSBs.
- HRT with VSBs during the winter period may lead to non-cost-effective systems at flow rates over 200 $m^3/d$, according to medium availability (NovaTec Consultants Inc., 1996).

## A.3 Combined Suspended-Growth and Attached-Growth Nitrification Processes

### *A.3.1 Dual Biological Processes*

#### A.3.1.1 Process Considerations

The dual biological process arrangement for enhanced total ammonia-nitrogen control may be installed as an upgrade measure, where either an existing trickling filter or activated sludge process is complemented by implementation of the other process type for enhanced nitrification capability. A wide variety of upgrade options are available, permitting customization of the dual process to the effluent quality and capacity requirements of a particular application, giving consideration to the size and characteristics of the existing facility (Daigger and Buttz, 1998). The first-stage attached-growth process is operated at relatively high loading rates for cBOD removal to facilitate nitrification in the second-stage suspended-growth process.

### *A.3.2 Hybrid Biological Processes*

#### A.3.2.1 Process Considerations

The amount of medium required and the placement of the medium within the wastewater treatment plant process depend on the influent wastewater composition and treatment objectives and on the design, configuration and operational characteristics of the existing biological treatment process. The surface areas available for attached growth with the various types of media, on a total reactor volume basis, include 65–150 $m^2/m^3$ for rope, 75–250 $m^2/m^3$ for sponge, 150–350 $m^2/m^3$ for plastic carrier elements and 35–10 $m^2/m^3$ for fixed plastic media (WERF, 2000). Fill levels of certain plastic carrier elements can be as high as 70% (v/v) based on the empty aeration tank volume. Specific nitrification rates of the attached growth are reportedly 0.4–1.0 g ammonium-nitrogen/$m^2$ per day for free-moving media and 0.2–0.6 g/m per day for rope-type media, corresponding to a process temperature of 10–12°C (WERF, 2000).

The aeration system of existing process aeration tank(s) may require modification to satisfy the increased oxygen demand associated with endogenous respiration of the elevated biomass content, requirements for nitrification and requirements for the maintenance of DO concentration of at least 3 mg/L required by the fixed-film process. Furthermore, depending on the type of medium used, there may be specific requirements for the type and placement of aeration devices to promote proper aeration and mixing patterns. It is essential that effective mass transfer and aerobic conditions are maintained within the medium zone to minimize diffusional resistances of the biofilm.

The use of free-moving media requires the installation of retention screens at the inlet and outlet of the media zones. These screens should be equipped with air knives to minimize the potential for fouling and plugging. Furthermore, sponge-type media require submersible airlift pumps to squeeze media to remove excess biomass and contaminants.

The location of the medium zone can influence the specific nitrification activity and corresponding medium surface area requirements, particularly in the retrofitting of a lagoon process. For instance, media placed in proximity to the lagoon inlet would benefit from increased wastewater temperatures. Alternatively, a medium zone located further downstream in the lagoon process would experience more favourable wastewater cBOD:TKN ratios but would be subject to lower wintertime process temperature effects.

Another option for the upgrading of an existing conventional activated sludge plant is to place the medium in the up-front zones of the aeration tank to promote heterotrophic biofilm growth and increase cBOD removal capacity. This would provide a more favourable wastewater cBOD:TKN condition in the downstream portion of the activated sludge process to facilitate nitrification by the suspended-growth biomass. Excessive biofilm growth is a potential problem associated with this arrangement. This process concept is being implemented on the full scale at the Peterborough, Ontario, water pollution control plant.

### A.3.2.2 Advantages/Limitations

Some of the advantages to an hybrid system include the relative ease of operation and the speed at which a retrofit can be implemented. For example, apart from ensuring that adequate aeration is available and that retention screens are kept clean, there are no significant operational changes required for an IFAS process compared with a conventional activated sludge facility. The existing facilities and tankage can be utilized. Conversely, a more conventional plant expansion involving the construction of additional aeration basins and secondary clarifiers requires many months to design, permit, bid, construct and commission. The design engineering phase for a hybrid process retrofit is much less complex and consists of determining the amount of media required and their placement. Since the operation and monitoring requirements of the IFAS and conventional activated sludge processes are essentially the same, conversion to a hybrid design is a relatively simple upgrade measure.

Other benefits associated with the hybrid process include the following:

- A low-cost nitrification upgrade is an option, compared with conventional process expansion.
- The hybrid process offers increased stability compared with conventional activated sludge processes owing to reduced susceptibility to losses of biomass and nitrifying activity.
- Combined attached-growth and acclimated suspended-growth biomass offers increased process flexibility and control and reduced media surface area and volume requirements than for the MBBR process described in Section 5.1.4.
- There is potential for lower design mixed liquor SRT for nitrification, with concomitant reductions in solids loading rates to the secondary clarification stage.
- Free-moving rigid plastic carrier elements possess a high degree of durability and specific surface area, offer good mass transfer characteristics and, in the case of coarse-bubble aeration, can enhance oxygen transfer efficiency.
- Frames supporting rope-type media are relatively simple to remove from the aeration tank, if necessary (e.g., for maintenance of aeration tank).

- IFAS processes can be readily upgraded to biological nutrient removal process configurations, if necessary.

Some potential limitations associated with the hybrid process and support media include the following:

- Existing aeration tank(s) possibly have insufficient volume to accommodate the required medium surface area and volume (e.g., for high-rate activated sludge processes, elevated influent TKN conditions and/or low specific nitrification rates).
- Rope-type media systems may be susceptible to anaerobic zones and the growth of predators on the media if not properly designed and operated.
- Sponge-type media are susceptible to wear and deterioration and must be periodically replaced.
- Free-moving plastic carrier elements can sometimes bunch up at the discharge end of the aeration tank, particularly at high medium fill levels.
- Use of powdered activated carbon as carrier material is expensive and requires a complex medium regeneration system and continuous makeup to compensate for losses.
- Modification of the aeration system or the installation of a new aeration system may be required.
- There are only a limited number of full-scale applications of hybrid treatment systems.

## A.4 Other Biological Processes for Ammonia Removal

### *A.4.1 Facultative Lagoons*

### A.4.1.1 Process Considerations

Conventional lagoon systems are capable of providing secondary treatment at considerably lower costs than mechanical sewage treatment processes. This is especially the case for small communities, where sufficient low-cost land is available in the vicinity of the service area. Conventional lagoon systems can also provide wastewater retention capabilities for annual discharge. The latter is an important consideration in cold-climate applications, for which sufficient storage capacity is required to permit a once-per-year period of treated effluent discharge in the autumn, with total ammonia-nitrogen concentrations of <5 mg/L (Alberta Environment, 1987). Alberta Environment recommends that storage ponds have a maximum depth of 2.5 m and a minimum storage period of 1 year (NovaTec Consultants Inc., 1996).

Facultative lagoon effluent quality during the wintertime and springtime would not be expected to meet the target total ammonia-nitrogen level of <5 mg/L. Reported total ammonia-nitrogen discharge concentrations typically measure between 6 and 20 mg/L (Alberta Environment, 1987; OMOE and Environment Canada, 1993). These elevated releases are due to a low potential degree of treatment during the wintertime, snow and ice cover, turnover of pond contents in the springtime, and the concomitant release of anaerobic decomposition by-products such as ammonium from the benthal zone (i.e., sludge deposits) at the onset of warmer temperature conditions. The complex chemical reactions associated with sludge decomposition in the benthal zone can result in a relatively low degree of control over treatment process performance and effluent quality compared with the completely mixed aerated lagoon process.

Technological options for the posttreatment of lagoon effluents to achieve year-round nitrification in cold-climate applications are detailed in Section 6.1.3. Additional information on design and operational aspects, full-scale experiences and performance characteristics with conventional facultative lagoon processes can be found in

USEPA (1983, 1992), Alberta Environment (1987), Metcalf & Eddy (1991) and NovaTec Consultants Inc. (1996).

### A.4.1.2 Advantages/Limitations

The main advantage of the conventional facultative lagoon is the simplicity of O&M and the relatively low associated capital and O&M costs in comparison with mechanical secondary treatment processes. Sludge disposal requirements and costs are relatively low. However, land area requirements are high, especially where long-term effluent storage is provided for annual autumn discharge.

Conventional lagoon systems have significantly reduced treatment capabilities, including ammonia removal, during winter conditions in cold-climate locations such as Canada and cannot consistently meet the secondary treatment capabilities of mechanical secondary treatment processes (Environment Canada, 1987) unless long-term effluent storage capacity or an add-on treatment process stage is provided. The latter upgrade option would permit continuous-discharge operations but would significantly increase system complexity and cost.

## A.5 Physical/Chemical Processes for Ammonia Removal

Physical/chemical removal of total ammonia-nitrogen can involve conversion into nitrogen gas, stripping of un-ionized ammonia, ion exchange or membrane separation. This can be accomplished by the following technologies:

- breakpoint chlorination;
- air stripping;
- ion exchange; and
- membrane separation (e.g., reverse osmosis).

## A.5.1 Breakpoint Chlorination

### A.5.1.1 Process Considerations

The breakpoint chlorination process is relatively difficult to control and does not represent a practical solution except as an effluent polishing technique (WEF and ASCE, 1992). Sufficient chemical must be added to ensure the virtually complete removal of total ammonia-nitrogen. It has been recommended that the process be considered only for occasional polishing of biologically treated effluents, where necessary, to meet stringent total ammonia-nitrogen discharge limits (USEPA, 1993) or as an emergency backup process (WEF, 1998). Another application could be the polishing of effluents from continuous-discharge aerated lagoons during seasonal low temperature conditions, where the biological process suffers from reduced ammonia removal efficiency. An example of such an application is the Niagara-on-the-Lake, Ontario, lagoon-based water pollution control plant, which seasonally employs breakpoint chlorination for short periods of time during the wintertime.

Owing to the high reactivity of chlorine, undesirable by-products can be generated. These include chlorinated organic substances, mainly trihalomethanes such as chloroform. Chlorinated organic compounds are recognized as potential carcinogens.

### A.5.1.2 Advantages/Limitations

The following are advantages of the breakpoint chlorination process:

- relatively low associated capital cost; and
- unlike biological processes, the ability to be shut down for prolonged periods and then quickly restarted whenever required.

Process limitations include the following:

- difficult to control;
- depletion of alkalinity during chlorination and dechlorination;

- possible requirement for supplemental alkalinity to maintain satisfactory pH levels for reasonable chlorination reaction rates;
- generation of undesirable chlorinated organic by-products; and
- health and safety concerns associated with the storage and dispensing of chlorine gas.

## A.5.2 Ion Exchange

### A.5.2.1 Process Considerations

The selective ion exchange process for ammonia removal from wastewater is relatively expensive but best suited for applications that experience prolonged periods of freezing conditions yet where very high degrees of removal must be consistently maintained (USEPA, 1971). The application of ion exchange in wastewater treatment requires filtration as an essential pretreatment step to remove TSS and avoid clogging of the resin. The filtered wastewater is then fed to the resin column or bed. The effluent ammonium-nitrogen concentration from the passage through the resin gradually increases from 0.5 to 1.0 mg/L, as the ion exchange sites become saturated. The effluent ammonium-nitrogen progressively deteriorates up to a level of 5.0 mg/L, at which point regeneration of the resin may be initiated according to the effluent limits permitted. The wastewater liquid volumes that can be passed through the resin to reach an effluent ammonium-nitrogen concentration of 1.0 mg/L and 5.0 mg/L are reported as 130 and 170 column or bed volumes, respectively (USEPA, 1993).

Regeneration of the resin is typically achieved by passage of a high-pH salt solution through the column or bed. The regenerant volumes required vary from 10 to 20 column or bed volumes. The end cycle of regeneration is followed with a rinse cycle to reduce pH and ammonia concentration in the startup effluent of the next ion exchange cycle. An ammonia stripping process may then be used to recover the regenerant solution for reuse. The pH of the regenerant is adjusted to 11 or higher with caustic and then enters a stripping cycle of the un-ionized ammonia. The off-gases are then absorbed in a sulphuric acid solution to form ammonium sulphate, which can be used as a nitrogen supplement for organic fertilizers. Other innovative means of regenerant recovery are available.

Clinoptilolite is a naturally occurring ion exchange zeolite resin that possesses high selectivity for ammonium. The clinoptilolite has shown a range of 0.2–0.5 meq/L total exchange capacities for ammonium-nitrogen in wastewater treatment applications (USEPA, 1993). Two wastewater treatment plant applications have used the clinoptilolite ion exchange properties.

The 19 MLD North Tahoe-Truckee Wastewater Treatment Plant and 57 MLD Upper Occoquan Treatment Facility have used ion exchange technology with clinoptilolite resin. The plants respectively achieved ammonium-nitrogen concentrations of 2.5 mg/L and 2 mg/L in the ion exchange effluent. In the case of North Tahoe, the concentration of ammonium-nitrogen was further reduced to 0.3 mg/L using breakpoint chlorination. Effluent total ammonia-nitrogen requirements for North Tahoe have been relaxed, and the plant is achieving an average ammonium-nitrogen concentration of 5–6 mg/L without breakpoint chlorination (USEPA, 1993).

### A.5.2.2 Advantages/Limitations

Ion exchange may be used to produce a treated effluent with low residual total ammonia-nitrogen concentration (i.e., 0.5–1.0 mg/L), but the technology appears to be operationally demanding. The process is applicable under cold temperature conditions.

Clinoptilolite resin may require replacement by up to 20% per year and is expensive. The ammonia stripping system requires regular hydrochloric acid washing. The acid and caustic solutions used in the resin regeneration and ammonia stripping are corrosive and require costly equipment to work in this environment. High capital and O&M costs render this technology a relatively expensive option for effluent tertiary treatment (USEPA, 1971).

## A.5.3 Air Stripping

### A.5.3.1 Process Considerations

Ammonia stripping requires an effluent pH of at least 10.5 units to raise the fraction of un-ionized ammonia. This may require supplemental lime addition. This process requires the capture of off-gases for further treatment or chemical recovery.

### A.5.3.2 Limitations

Limitations of the air stripping process include the following:

- costlier than biological nitrification;
- operation of air stripping towers is effectively limited to wastewater temperatures of greater than 10°C;
- potential for scaling problems;
- reduced ammonia removal efficiencies with scale formation;
- possible alkalinity requirement for pH adjustment; and
- high energy requirement for supply of air, typically at an air-to-liquid volume ratio of about 50:1.

## A.5.4 Reverse Osmosis

### A.5.4.1 Process Considerations

Pretreatment is required with reverse osmosis application to maximize the efficiency and life of the membrane process. The pretreatment applications are aimed at minimizing scaling, fouling and degradation of the membranes, thus optimizing the salt and molecule separation process and operating costs (WEF and ASCE, 1998).

Scaling effect by precipitation of soluble salts, such as calcium, may be prevented with the use of acid addition to solubilize calcium carbonate, softening with a strong acid cation exchange resin to remove scale-forming cations and replace by sodium cations, and lime softening to remove carbonate hardness. Preventative short-cycle cleaning of the membrane may control scaling impacts.

Fouling of the membrane by colloids, including clay, silica, iron corrosion products, alum and ferric chloride not removed in clarifiers, may be controlled with wastewater filtration through granular media, conventional coagulation-flocculation, and in-line coagulation and flocculation followed by filtration. Microfiltration or ultrafiltration can be used to remove all suspended matter. Dissolved organic compounds can be removed by ultrafiltration systems according to their molecular mass.

Reverse osmosis processes typically employ tubular membrane or spiral wound systems. The tubular configuration consists of a porous wall tube with the membrane inserted on the inner wall. The wastewater flow is axial within the tube, and the permeate flow is radial through the membrane and the porous structure. The tubes are arranged in bundles, and each unit bundle is called a module. All tubes of the module are connected in series, so the module has only one wastewater entrance and exit for the concentrate flow. The spiral wound configuration consists of a spiral envelope made from a membrane with a porous material inside the envelope. The feed wastewater is applied outside the envelope; the permeate flow goes through the membrane, is collected in the porous material within the envelope and is evacuated through a collection tube. The concentrate flow is evacuated at the end-side of the envelope.

### A.5.4.2 Advantages/Limitations

Reverse osmosis processes may be used for the tertiary treatment of effluents and generation of ultra-high-purity effluent in water reclamation applications. The technology has shown a satisfying removal efficiency for ammonia.

Reverse osmosis requires significant levels of pretreatment and is intensively energy consuming. Furthermore, the capital cost for the technology, especially for medium to high flow rate applications, would be very high and would likely preclude its use for effluent tertiary treatment.

## Table A4 Biological Suspended-Growth Ammonia Removal Processes and Identified Vendors

| Process Classification | Process | Vendor (Process Name) | Vendor Contacts (Telephone No.) |
|---|---|---|---|
| Biological suspended growth<br>- Mechanical plants | Activated sludge (e.g., conventional, complete mix, plug flow, step feed, extended aeration, contact stabilization, biological nutrient removal, step feed biological nutrient removal; single-stage or two-stage) | Engineering Process & Design Consultants | |
| | | *Biological nutrient removal plants also supplied by:* | |
| | | ONDEO Degrémont (*OCO*) | Jean Marchand: 514-683-1200 |
| | | Parkson (*Biolac, Wave Oxidation*) | Jean Grenier: 514-636-8712 |
| | | Smith & Loveless (*Model RE, NUTRIGEST*) | |
| | | USFilter / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | | Waterlink (*Sequox*) | |
| | High-purity-oxygen activated sludge | Krüger (OASES) / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | | Lotepro Corp. (*UNOX, InNitri® sidestream process*) | |
| | Oxidation ditch | Krüger (*A/O, A2/O, BioDenitro, BioDenipho*) / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | Sequencing batch reactor (SBR) | ABJ-Sanitaire (*ICEA*) | |
| | | ADI | Robert Landine: 506-452-7307 |
| | | Aqua Aerobic Systems | |
| | | CASS Water Engineering (*CASS*) | |
| | | ECO-Process and Equipment International | 450-477-1510 |
| | | Fluidyne Corp. (*H2FLOW*) | |
| | | Mixing Systems | |
| | | Sanitherm | |
| | | USFilter (*Jet Tech*) / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | | Waterlink | |
| | Membrane bioreactor (MBR) | Koch Membrane | |
| | | Kubota | |
| | | ONDEO Degrémont (*BRM*) | Jean Marchand: 514-683-1200 |
| | | OTV (*BIOSEP*) / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | | USFilter / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | | Zenon Environmental (*ZenoGem*) | Doug Thompson: 905-465-3030 |
| Biological suspended growth<br>- Low-rate plants<br>- Single-cell or multicell process configurations | Lagoon systems (i.e., aerated, facultative) | Engineering Process & Design Consultants | |
| | | *Proprietary lagoon processes supplied by:* | |
| | | ADI | Robert Landine: 506-452-7307 |
| | | Lemna (*LemTec, CLASS*) | Stacy Volk: 612-253-2002 |
| | | Parkson (*Biolac*) | Jean Grenier: 514-636-8712 |

## Table A5 Biological Attached-Growth Ammonia Removal Processes and Identified Vendors

| Process Classification | Process | Vendor (Process Name) | Vendor Contacts (Telephone No.) |
|---|---|---|---|
| Biological attached growth | Trickling filter | Eimco | |
| | | Mass Transfer | A. Derek/ R. Frank:<br>+44(0)1539 563851 |
| | | NSW | |
| | | Sanitherm | |
| | | USFilter (*Envirex*) / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | Rotating biological contactor (RBC, including SBC) | CMS Group | |
| | | Klargester | |
| | | NSW | |
| | | OxydH2O | |
| | | Sanitherm | |
| | | USFilter (*Envirex*) / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | Biological aerated filter (BAF) | Krüger (*BIOSTYR; upflow-cocurrent air; one or two stages*) / [John Meunier Inc.] | Christian Scott: 514-334-7230 |
| | | ONDEO Degrémont (*Biofor; upflow - cocurrent air*) | Jean Marchand: 514-683-1200 |
| | Biological fluidized-bed reactor | OTV (*BIOLIFT*) / [John Meunier Inc.] | Patrick Binot (OTV Paris) /<br>Christian Scott: 514-334-7230 |
| | Fixed/packed-bed biofilm reactor | NSW | |
| | | Smith & Loveless (*FAST*) | |
| | Moving-bed biofilm beactor (MBBR) | ANOX (*Natrix*) | |
| | | Hydroxyl | Juergen Puetter: 250-655-3348 |
| | | Kaldnes Miljoteknologi (*Kaldnes*) | Chandler Johnson: 401-270-3898 |
| | Constructed wetlands | Engineering Process & Design Consultants | |
| | Intermittent or recirculating sand filters | Engineering Process & Design Consultants | |
| | | *Proprietary lagoon processes supplied by:* | |
| | | Orenco Systems | Matt Converse: 1-800-348-9843 |

### Table A6 Hybrid Biological Ammonia Removal Processes and Identified Vendors

| Process Classification | Process | Vendor (Process Name) | Vendor Contacts (Telephone No.) |
|---|---|---|---|
| Hybrid biological (suspended-growth and attached-growth biomass)<br>- Integrated fixed-film activated sludge (IFAS)<br>- Retrofitted lagoons | Rope media | BioMatrix<br>Ringlace | John Haley III: 401-941-9280<br>John Hunington: 503-251-1295 |
| | Sponge media | Captor<br>Lotepro (*Linpor*) | Dick Johansen: 914-747-3500 |
| | Moving-bed plastic media | ANOX (*Natrix*)<br>ONDEO Degrémont (PEGAZUR) [ONDEO Degrémont - Paris, France]<br>Kaldnes Miljoteknologi (*Kaldnes*) | Jean-Claude Bourdelot: +33 (1) 46 25 6200<br>Chandler Johnson: 401-270-3898 |
| | Fixed plastic media | Lemna (*LPR*)<br>NSW<br>Smith & Loveless (*FAST, Floating FAST*)<br>Tropac | Stacy Volk: 612-253-2002 |
| | Powdered activated carbon | USFilter (Zimpro; PACT) / [John Meunier Inc.] | Christian Scott: 514-334-7230 |

## Table A7 Physical/Chemical Ammonia Removal Processes and Identified Vendors

| Process Classification | Process | Vendor (Process Name) | Vendor Contacts (Telephone No.) |
|---|---|---|---|
| Physical/chemical | Ion exchange | ONDEO Degrémont | Jean Marchand: 514-683-1200 |
| | | Osmonics | |
| | Breakpoint chlorination | Engineering Process & Design Consultants | |
| | Air stripping | Sanitherm | |
| | Reverse osmosis | ONDEO Degrémont | Jean Marchand: 514-683-1200 |
| | | Koch | |
| | | Osmonics | |
| | | Zenon Environmental | Doug Thompson: 905-465-3030 |

# Appendix B
# (Summary of Inhibitory Substances)

**Table B1** **Inhibition of Nitrification by Inorganic and Organic Chemicals (WEF, 1998)**

| Substance | Inhibitory Concentration (mg/L) |
|---|---|
| **Metals** | |
| Cadmium | 14.3 |
| Chromium (III) | 10 |
| Copper | 230 |
| Lead | 0.50 |
| Nickel | 5 |
| Zinc | 11 |
| **Other Inorganics** | |
| Ammonia (un-ionized) | 5 |
| Ammonium (see Figure 4.3) | 1000 |
| Carbon disulphide | 35 |
| Chlorine | 1 |
| Cyanide | 16 |
| Hydrazine | 58 |
| Hydrazine sulphate | 200 |
| Hydrogen sulphide | 50 |
| Nitrite (see Figure 4.3) | 100 |
| Potassium chlorate | 2500 |
| Potassium chromate | 800 |
| Potassium dichromate | 6.0 |
| Sodium arsenite | 2000 |
| Sodium azide | 20 |
| Sodium chloride | 35 000 |
| Sodium fluoride | 1220 |
| Sulphides | 5 |
| **Organics** | |
| Acetone | 2000 |
| Allyl alcohol | 20 |
| Allyl chloride | 180 |
| Allyl isothiocyanate | 1.9 |
| Allyl thiourea | 1.2 |
| AM (2-amino-4-chloro-6-methylpyrimidine) | 50 |
| Amino acids | 1-1000 |
| Aminoethanol | 12 |
| Aminoguanidine | 74 |
| 2-Aminophenol | 0.27 |
| 4-Aminophenol | 0.07 |
| Aminopropiophenone | 43 |
| Aminotriazole | 70 |
| Aniline | 7.7 |
| 1-Arginine | 1.7 |
| Benzene | 13 |
| Benzidine dihydrochloride | 50 |
| Benzocaine | 100 |
| Benzothiazole disulphide | 38 |
| Benzylamine | 100 |
| Benzyldimethyldodecylammonium chloride | 2 |
| Benzylthiuronium chloride | 40 |
| 2,2'-Bipyridine | 10 |
| Bisphenol A | 100 |
| Bromodichloropropane | 84 |

## Table B1 Inhibition of Nitrification by Inorganic and Organic Chemicals (WEF, 1998) *Contd.*

| Substance | Inhibitory Concentration (mg/L) |
|---|---|
| 2-Bromophenol | 0.35 |
| 4-Bromophenol | 0.83 |
| *n*-Butanol | 8200 |
| Carbamate | 2 |
| Chlorobenzene | 0.71–500 |
| Chloroform | 18 |
| 2-Chloronaphthol | 14 |
| 2-Chlorophenol | 2.7 |
| 3-Chlorophenol | 0.20 |
| 4-Chlorophenol | 0.73 |
| 5-Chloro-1-pentyne | 0.59 |
| 2-Chloro-6-trichloromethyl-pyridine | 11 |
| *m*-Cresol | 1–100 |
| *o*-Cresol | 11 |
| *p*-Cresol | 13 |
| Cyclohexylamine | 0.50 |
| Diallyl ether | 100 |
| Dibromoethane | 60 |
| 1,2-Dibromoethane | 50 |
| 1,2-Dichlorobenzene | 100 |
| 1,1-Dichloroethane | 0.91 |
| 2,4-Dichloroethane | 0.79 |
| 1,5-Dichloropentane | 13 |
| 2,3-Dichlorophenol | 0.5 |
| 2,6-Dichlorophenol | 8.1 |
| 3,5-Dichlorophenol | 3.0 |
| 1,3-Dichloropropene | 0.6 |
| Dicyandiamide | 250 |
| Dicyclohexylcarbodiimide | 10 |
| Diethyl dithiothiosemicarbazide | 0.1 |
| Diguanide | 50 |
| Dimethylgloxime | 140 |
| Dimethylhydrazine | 19 |
| Dimethyl *p*-nitrosoaniline | 25 |
| 2,4-Dinitrophenol | 37 |
| Diphenylthiocarbazone | 7.5 |
| Dithio-oxamide | 1 |
| Dodecylamine | <1 |
| Erythromycin | 50 |
| Ethanol | 2400 |
| Ethanolamine | 100 |
| Ethyl acetate | 18 |
| Ethylenediamine | 100 |
| Ethyl urethane | 1000 |
| Ethyl xanthate | 10 |
| Flavonoids | 0.01 |
| Guanidine | 4.7 |
| Hexamethylene diamine | 85 |
| Histidine | 5 |
| 8-Hydroxyquiniline mercaptobenzothiazole | 1 |
| Lauryl benzenesulphonate | 120 |
| 1-Lysine | 4 |
| Mercaptobenzothiazole | 3 |
| Methanol | 160 |
| Methionine | 9 |
| Methylamine hydrochloride | 100 |
| *n*-Methylaniline | <1 |
| Methylene blue | 30 |
| Methylhydrazine | 12 |
| Methyl isothiocyanate | 0.80 |
| Methyl mercaptan | 300 |

**Table B1** **Inhibition of Nitrification by Inorganic and Organic Chemicals (WEF, 1998)** ***Contd.***

| Substance | Inhibitory Concentration (mg/L) |
|---|---|
| Methylpyridines | 100 |
| 2-Methylpyridines | 100 |
| 4-Methylpyridines | 100 |
| Methylthiourea | 0.45 |
| Methyl thiuronium sulphate | 1 |
| Monethanolamine | >200 |
| N-Serve (Nitrapyrin) | 10 |
| Naphthylethylenediamine dihydrochloride | 23 |
| Ninhydrin | 10 |
| *p*-Nitroaniline | 10 |
| *p*-Nitrobenzaldehyde | 50 |
| Nitrobenzene | 50 |
| 2-Nitrophenol | 50 |
| 3-Nitrophenol | 11 |
| 4-Nitrophenol | 2.6 |
| Nitrourea | 1.0 |
| Panthothenic acid | 50 |
| Pentachloroethane | 7.9 |
| Perchloroethylene phenol | 5.6 |
| Phenolics (substituted) | 100 |
| Phenolic acids | 0.01 |
| *p*-Phenylazoaniline | 100 |
| Potassium thiocyanate | 300 |
| *n*-Propanol | 20 |
| Purines | 50 |
| Pyridine | 10 |
| Pyrimidines | 50 |
| Pyruvate | 400 |
| Resorcinol | 7.8 |
| Skatole | 7 |
| Sodium cyanate | 100 |
| Sodium cyanide | 1 |
| Sodium dimethyl dithiocarbamate | 14 |
| Sodium methyl dithiocarbamate | 1 |
| ST (sulphathiazole) | 50 |
| Strychnine | 100 |
| Tannin | 0.01 |
| Tetrabromobisphenol | 100 |
| 1,2,3,4-Tetrachlorobenzene | 20 |
| 1,2,4,5-Tetrachlorobenzene | 9.8 |
| 1,1,1,2-Tetrachloroethane | 8.7 |
| 1,1,2,2-Tetrachloroethene | 1.4 |
| 2,3,5,6-Tetrachlorophenol | 1.3 |
| Tetramethylammonium chloride | 2200 |
| Tetramethyl thiuram disulphide | 5 |
| Thiamine | 0.53 |
| Thioacetamide | 500 |
| Thiocyanates | 0.18 |
| Thiosemicarbazide (aminothiourea) | 0.76 |
| Thiourea | 1 |
| Thiourea (substituted) | 3.6 |
| Threonine | 50 |
| 1-Threonine | 5 |
| 2,4,6-Tribromophenol | 2.5–50 |
| 1,1,2-Trichloroethane | 1.9 |
| 2,2,2-Trichloroethanol | 2.0 |
| Trichloroethylene | 0.81 |
| Trichlorophenol | 100 |
| 2,3,5-Trichlorophenol | 3.9 |
| 2,3,6-Trichlorophenol | 0.42 |
| 2,4,6-Trichlorophenol | 7.9 |

**Table B1** Inhibition of Nitrification by Inorganic and Organic Chemicals (WEF, 1998) *Contd.*

| Substance | Inhibitory Concentration (mg/L) |
|---|---|
| Triethylamine | 100 |
| Trimethylamine | 120 |
| 2,4,6-Trimethylphenol | 30 |
| 1-Valine | 1.8 |
| Vitamins (riboflavin, A-lipolic acid, B-pyridoxine HCL) | 50 |

# Appendix C (Glossary)

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| **activated sludge** | The biologically active solids in an activated sludge process wastewater treatment plant. |
| **activated sludge process** | A biological wastewater treatment process in which a mixture of wastewater and biologically enriched sludge is mixed and aerated to facilitate aerobic, anoxic or anaerobic decomposition by microbes. |
| **aeration** | The addition of air or oxygen to water or wastewater, usually by mechanical means, to increase dissolved oxygen levels and maintain aerobic conditions. |
| **aerobic processes** | Processes in wastewater treatment systems that take place in the presence of dissolved oxygen. |
| **air diffuser** | A device designed to transfer atmospheric oxygen into a liquid. |
| **air scour** | The agitation of granular filter media with air during the filter backwash cycle. |
| **air stripping** | The process of removing volatile and semivolatile, such as molecular nitrogen, contaminants from liquid by passing air and liquid countercurrently through a packed tower. |
| **algae** | Single-celled to multicelled organisms that rely on photosynthesis for growth. Most algae are classified as plants. |
| **algal blooms** | Rapidly reproducing floating colonies of algae that can cover streams, lakes and reservoirs, creating nuisance conditions. |
| **alkalinity** | The ability of a water to neutralize an acid through the presence of carbonate, bicarbonate and hydroxide ions. |
| **ammonia** | A compound of hydrogen and nitrogen, also referred to as free ammonia or un-ionized ammonia. Chemical formula is $NH_3$. |
| **ammonification** | Bacterial decomposition of organic nitrogen compounds to ammonia forms ($NH_4^+/NH_3$). |
| **ammonium** | The ionized form of ammonia found in solution, also referred to as ionized ammonia. Chemical formula is $NH_4^+$. |
| **anaerobic digestion** | Sludge stabilization process in which the organic material in biological sludges is converted to methane and carbon dioxide in an airtight reactor. |
| **anaerobic processes** | Processes in wastewater treatment systems that take place in the absence of dissolved oxygen and nitrates. |

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| **anoxic processes** | Processes in wastewater treatment systems that take place in the absence of dissolved oxygen. |
| **atmospheric reaeration** | Introduction of atmospheric oxygen into the water at the water's surface, which provides dissolved oxygen to the wastewater treatment environment. |
| **attached-growth process** | Biological wastewater treatment process whereby the microbes responsible for conversion of the organic matter in wastewater are attached to an inert medium such as rock or plastic materials. Also called *fixed-film* process. |
| **automatic sampling** | Collection of samples of prescribed volume over a defined time period by an apparatus designed to operate remotely without direct manual control. |
| **autotrophic** | Types of reactions that generally require only inorganic reactants; for example, nitrification. |
| **autotrophic microorganism** | Organism that derives its cell carbon from inorganic carbon dioxide. |
| **backwash** | A high-rate reversal of flow for the purpose of cleaning or removing solids from a filter bed or screening medium. |
| **bioassay** | An analytical method that considers a change in biological activity as a means of analyzing a material's response to biological treatment or an environment. |
| **biochemical oxygen demand (BOD)** | Dissolved oxygen demand that decomposition of organic matter places on a wastewater treatment process. BOD, as expressed in milligrams per litre (mg/L), is used as a measure of wastewater organic strength and as a measure of treatment performance. |
| **biodegradable** | Term used to describe organic matter that can undergo biological decomposition. |
| **biofilm** | An accumulation of microbial growth on the surface of a given medium. |
| **biological process** | The process by which the metabolic activities of bacteria and other microorganisms break down complex organic materials to simple, more stable substances. |
| **biomass** | The mass of biological material contained in a system. |
| **breakpoint chlorination** | Addition of chlorine until the chlorine demand has been satisfied for disinfection purposes and oxidation of ammonium-nitrogen to molecular nitrogen gas. |
| **bulking sludge** | A poorly settling activated sludge that results from the predominance of filamentous organisms. |

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| **calcium carbonate equivalent (mg/L as $CaCO_3$)** | A convenient unit of exchange for expressing all ions in water by comparing them with calcium carbonate, which has a molecular weight of 100 and an equivalent weight of 50. |
| **carbon dioxide** | A non-combustible gas formed in animal respiration and the combustion and decomposition of organic matter, serves as a carbon source for autotrophic organism cell synthesis. Chemical formula is $CO_2$. |
| **carbonaceous biochemical oxygen demand** | The portion of biochemical oxygen demand whereby oxygen consumption is caused by oxidation of carbon without nitrification oxygen demand, usually measured after a sample has been incubated for 5 days. |
| **chemical oxygen demand** | Measurement of organic matter in a water or wastewater that can be oxidized by using a chemical oxidizing agent. |
| **chemical treatment** | Any water or wastewater treatment process involving the addition of chemicals to obtain a desired result, such as precipitation, coagulation, flocculation, sludge conditioning, disinfection or odour control. |
| **clarification** | Any process or combination of processes whose primary purpose is to reduce the concentration of suspended matter in a liquid. |
| **clarifier** | A quiescent tank used to remove settleable suspended solids by gravity settling. Also called *sedimentation basins* or *settling basins*, they are usually equipped with a motor-driven chain-and-flight or rake mechanism to collect settled sludge and move it to a final removal point. |
| **coagulation** | The initial aggregation of finely divided suspended solids by the addition of a polymer or a biological process. |
| **coarse-bubble aeration** | An aeration system that uses submerged diffusers that release relatively large bubbles. |
| **composite sample** | A combination of individual samples of water or wastewater taken at preselected intervals to minimize the effect of variability of individual samples. |
| **concentration** | The amount of a substance dissolved or suspended in a unit volume of solution. |
| **degradation** | The biological breakdown of organic substances. |
| **denitrification** | Biological process in which nitrates are converted to molecular nitrogen gas. |
| **diffused-air aeration** | The introduction of compressed air to water by means of submerged diffusers or nozzles. |

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| **digestion** | The biological oxidation of organic matter in sludge, resulting in stabilization. |
| **dissolved oxygen** | The oxygen dissolved in a liquid. |
| **domestic wastewater** | Wastewater originating from sanitary conveniences in residential dwellings, office buildings and institutions. Also called *sanitary wastewater*. |
| **dynamic model** | Equation or set of equations that predict the time-varying performance of a process. Dynamic models are usually differential equations. Variables in the equations represent inputs, outputs and internal states of the process. |
| **effluent** | Partially or completely treated water or wastewater flowing out of a basin or treatment plant. |
| **endogenous respiration** | Bacterial growth phase during which microbes metabolize their own protoplasm without replacement because of low concentrations of available food. |
| **equalization** | The process of dampening hydraulic or organic variations in a flow so that nearly constant conditions can be achieved. |
| **eutrophication** | Nutrient enrichment of water causing excessive growth of aquatic plants and eventual deoxygenation of the water body. |
| **facultative microorganisms** | Microbes with the ability to survive with or without the presence of oxygen. |
| **filtrate** | Liquid remaining after removal of solids through filtration. |
| **fine-bubble aeration** | Method of diffused aeration using fine bubbles to take advantage of their high surface areas to increase oxygen transfer rates. |
| **fine screen** | A screening device usually having openings less than 6 mm. |
| **fixation** | Transformation of molecular nitrogen into organic and inorganic chemical forms assimilable by microorganisms. |
| **floc** | Small, gelatinous masses formed in water by adding a coagulant or in wastewater through biological activity. |
| **flocculation** | Gentle stirring or agitation following the coagulation step to accelerate the agglomeration of particles to enhance sedimentation or flotation. |
| **flotation** | A treatment process whereby gas bubbles are introduced to water and attach to solid particles, creating bubble-solid agglomerates that float to the surface, where they are removed. |

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| **flow equalization** | Transient storage of wastewater for release to a sewer system or treatment process at a controlled rate to provide a reasonably uniform flow. |
| **flow rate** | The volume or mass of a gas, liquid or solid material that passes some point in a stated period of time. |
| **food-to-microorganism ratio (F:M)** | The ratio of the influent biochemical oxygen demand or chemical oxygen demand to the volatile suspended solids concentration in a wastewater treatment aeration tank. |
| **grab sample** | A single water or wastewater sample taken at a time and place representative of total discharge. |
| **headworks** | The initial structure and devices located at the receiving end of a wastewater treatment plant. |
| **heavy metals** | Metals that can be precipitated by hydrogen sulphide in an acid solution and that may be toxic to microorganisms in excess of certain concentrations. |
| **heterotrophic microorganism** | Organism that derives its cell carbon from organic carbon. |
| **hydraulic loading** | Total volume of liquid applied per unit of time to a tank or treatment process. |
| **hydraulic retention time or hydraulic residence time** | Vessel volume (cubic metres) divided by the liquid removed (cubic metres per day, m$^3$/d). |
| **incineration** | The process of reducing the volume of a solid by burning of organic matter. |
| **industrial wastewater** | Liquid waste generated by manufacturing or industrial practices or processes. |
| **infiltration** | Water entering a sewer system through broken or defective sewer pipes, service connections or manhole walls. |
| **inflow** | Surface and subsurface water or stormwater discharged to a sewer system. |
| **influent** | Water or wastewater flowing to a basin or treatment plant. |
| **inhibitory compounds** | Inorganic or organic chemicals that slow or interfere with a chemical or biological reaction. |
| **innovative technology** | A process or technique that has not been fully proven under the circumstances of its contemplated use and that represents an advancement over the state of the art. |
| **instrumentation** | Use of technology to control, monitor or analyze physical, chemical or biological parameters. |
| **jet aeration** | Wastewater aeration system using floor-mounted nozzle aerators that combine liquid pumping with air diffusion. |

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| **leachate** | Fluid that percolates through solid materials or wastes and contains suspended or dissolved materials or products of the solids. |
| **liner** | A barrier of plastic, clay or other impermeable material that prevents leachate from contacting surface or subsurface water. |
| **lysis** | The rupture of a cell that results in loss of its contents. |
| **mass balance** | An analysis that delineates changes that take place in a reactor or system by quantifying system inputs and outputs. |
| **mechanical aeration** | The mechanical agitation of water to promote mixing with atmospheric air. |
| **membrane diffuser** | Fine-bubble aeration diffuser with perforated flexible plastic membranes. |
| **metabolism** | The biological conversion of organic matter and nutrients (total ammonia-nitrogen and phosphorus) to cellular matter and gaseous by-products. |
| **methanol** | A solvent often used as a supplemental carbon source during denitrification. Chemical formula is $CH_3OH$. |
| **microfiltration** | A low-pressure membrane filtration process that removes suspended solids and colloids larger than 0.1 micrometre (μm). |
| **microorganism** | Organisms observable only through a microscope. |
| **mixed liquor** | The mixture of wastewater and activated sludge undergoing aeration in the aeration basin. |
| **mixed liquor suspended solids** | Suspended solids in the mixture of wastewater and activated sludge undergoing aeration in the aeration basin. |
| **mixed liquor volatile suspended solids** | The volatile fraction of the mixed liquor suspended solids. |
| **model calibration** | Adjustment of model parameters by comparing the model predictions with those of a standard, such as instrument readings from actual plant operation. Adjustments are continued until the model predictions resemble the standard. |
| **molecular nitrogen** | Gaseous form of nitrogen ($N_2$) found in the atmosphere and the denitrification process end-products. |
| **municipal wastewater** | The liquid waste from residential, commercial, industrial and institutional sources. |
| **nitrate** | A stable, oxidized form of nitrogen having the formula $NO_3^-$. |
| **nitrification** | Two-step aerobic biological process in which total ammonia-nitrogen is converted first into nitrite and then into nitrate. |
| **nitrite** | An unstable, easily oxidized nitrogen compound with the chemical formula $NO_2^-$. |

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| **Nitrobacter** | Nitrifying bacteria that convert nitrites to nitrates, the second step of the nitrification process. Also called *nitrate formers.* |
| **nitrogen fixation** | The conversion of atmospheric molecular nitrogen to nitrogen compounds through biological activity. |
| **nitrogenous oxygen demand** | That portion of the oxygen demand associated with the oxygen consumption for the oxidation of nitrogenous material, usually measured after the carbonaceous oxygen demand has been satisfied. |
| **Nitrosomonas** | Nitrifying bacteria that convert ammonia to nitrite, the first step of the nitrification process. Also called *nitrite formers.* |
| **Nocardia** | Bacteria that can accumulate to create a nuisance foam in aeration basins and secondary clarifiers. |
| **nutrient** | Any substance that is assimilated by organisms to promote or facilitate their growth. |
| **organic loading** | The amount of organic matter applied to a treatment process. |
| **organic matter** | Substances containing carbon compounds, usually of animal or vegetable origin. |
| **organic nitrogen** | Nitrogen that is bound to carbon-containing compounds. |
| **oxic** | A biological environment that contains molecular oxygen. |
| **oxidation** | (1) A chemical reaction in which an element or ion loses an electron. (2) The biological or chemical conversion of organic matter to simpler, more stable forms. |
| **oxidation–reduction potential** | The potential required to transfer electrons from an oxidant to a reductant that indicates the relative strength potential of an oxidation–reduction reaction. |
| **oxygen transfer** | The exchange of oxygen between a gaseous and a liquid phase. |
| **oxygen transfer rate** | The mass of oxygen transferred per unit time. |
| **oxygen uptake** | The amount of oxygen used during biochemical oxidation. |
| **package plant** | Factory-assembled treatment plant generally incorporated in a single tank or, at most, a few tanks. |
| **pH** | The negative logarithm of the hydrogen ion concentration in gram moles per litre. On the 0–14 pH scale, a value of 7 at 25°C (77°F) represents a neutral condition. Decreasing values indicate increasing hydrogen ion concentration (acidity), and increasing values indicate decreasing hydrogen ion concentration (alkalinity). |

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| **photosynthesis** | Conversion of sunlight into organic matter by plants through a process of combining carbon dioxide and water in the presence of chlorophyll and light, which releases oxygen as a by-product. |
| **physical-chemical treatment** | Wastewater treatment processes that use only physical and chemical methods, such as filtration, coagulation/flocculation or sedimentation. |
| **phytoplankton** | Algae that are microscopic in size, which float or drift in the upper layer of the water column and depend on photosynthesis and the presence of phosphorus and nitrogen in the water. |
| **pilot plant** | A wastewater treatment plant that is smaller than full scale and used to test and evaluate a treatment process. |
| **pollutant** | A substance present in amounts that impair or threaten an ecosystem to the extent that its current or future uses are precluded. |
| **polymer** | A compound consisting of a chain of organic molecules used as coagulants or coagulant aids. |
| **preliminary treatment** | Treatment steps including comminution, screening, grit removal, preaeration and/or flow equalization that prepare wastewater influent for further treatment. |
| **pretreatment** | (1) The initial water or wastewater treatment process that precedes primary treatment processes. (2) The treatment of industrial wastes to reduce or alter the characteristics of the pollutants before discharge to a wastewater treatment plant. |
| **primary clarifier** | Sedimentation basin that precedes secondary wastewater treatment. Principal form of primary treatment to reduce the solids loading of subsequent treatment processes. |
| **primary effluent** | Product of primary treatment of wastewater that typically involves settling of solids in a containment structure. |
| **primary sludge** | Sludge produced in a primary waste treatment unit. |
| **primary treatment** | Settling/clarification process for removal of settleable solids from wastewaters. |
| **reactor** | The container or tank in which a chemical or biological reaction is carried out. |
| **receiving water** | Surface water body that receives effluent from a wastewater treatment plant. |
| **respiration** | Intake of oxygen and discharge of carbon dioxide as a result of biological oxidation. |

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| **respirometer** | An instrument used to study the character and extent of respiration. |
| **return activated sludge** | Settled activated sludge that is returned to mix with raw or primary settled wastewater. |
| **rhizome** | Root-like stem that produces roots and stems to propagate itself in a surrounding zone. |
| **runoff** | Rainwater, leachate or other liquid that drains over urban areas or agricultural land and reaches a drain, sewer or body of water. |
| **secondary clarifier** | A clarifier following a secondary treatment process and designed for gravity removal of suspended matter. |
| **secondary effluent** | Wastewater that has undergone secondary treatment and is discharged to the environment or receives further treatment in tertiary treatment processes. |
| **secondary treatment** | Continuity of the process begun in primary treatment by removing certain constituents, such as biochemical oxygen demand and total suspended solids, from primary effluent to prescribed treatment levels. |
| **septic** | The condition that results from biological degradation of organic matter in wastewater under anaerobic conditions, usually producing hydrogen sulphide or other odorous compounds. |
| **simulation** | The use of a computer program to predict the performance of a process under different conditions. Simulations are often employed when an actual system is difficult or costly to test. |
| **sloughing** | The disattachment of accumulated biological solids (biofilm) from attached-growth processes. |
| **sludge dewatering** | The removal of a portion of the water contained in sludge by means of a filter press, centrifuge or other mechanism. |
| **solids retention time or solids residence time or sludge age** | The mass of suspended solids or volatile suspended solids in reactors (in kilograms) divided by the solids removed (in kilograms per day) through wasted sludge and final effluent. Also referred to as mean cell residence time. |
| **steady state** | An equilibrium condition that exists in a system. |
| **stoichiometric** | The ratio of chemical substances reacting in water that corresponds to their combined weights in the theoretical chemical reaction. |
| **substrate** | The organic matter or nutrients that are used as food substances during biological wastewater treatment. |

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| **supernatant** | Liquid above the settled sludge layer in a sedimentation basin. |
| **surface loading rate** | A criterion used for design of attached-growth processes and sedimentation tanks expressed as flow per day per unit of medium or basin surface area. |
| **surface water** | Water from sources open to the atmosphere, including lakes, reservoirs, rivers, streams and marshes. |
| **suspended solids** | Solids captured by filtration through a glass wool mat or 0.45-micrometre (µm) filter membrane. |
| **suspended-growth process** | Biological wastewater treatment process in which the microorganisms responsible for the conversion of organic matter of the wastewater are maintained in suspension within the liquid. |
| **synthesis** | Assimilation of ammonium and nitrate by microorganisms to produce organic nitrogen compounds, such as proteins. |
| **tertiary filtration** | The use of a granular media filter to improve secondary effluent quality. |
| **tertiary treatment** | The use of physical, chemical or biological means to improve secondary effluent quality through removal of plant nutrients such as nitrogen and phosphorus, or lower $BOD_5$ and suspended solids effluent concentration to achieve local water quality standards. |
| **thickening** | A procedure used to increase the solids content of sludge by removing a portion of the liquid. |
| **total ammonia-nitrogen** | The combination of un-ionized and ionized ammonia. Can also be designated as total $NH_3$ N. |
| **total Kjeldahl nitrogen** | Organic nitrogen plus total ammonia-nitrogen. |
| **total nitrogen** | Sum of all the forms of nitrogen in wastewater, including nitrate, nitrite, ammonia and organic nitrogen. |
| **total phosphorus** | Measure of all forms of phosphorus in wastewater, including orthophosphate, polyphosphate and organic phosphate. |
| **total suspended solids** | The measure of particulate matter suspended in a sample of wastewater. After filtering a sample of a known volume, the filter is dried at 103–105°C and weighed to determine the residue retained. |
| **toxicity** | The property of being poisonous or causing an adverse effect on a living organism. |

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| **treatability study** | A study in which a waste is subjected to a treatment process to determine whether it is amenable to treatment and/or to determine the treatment efficiency or optimal process conditions for treatment. |
| **trihalomethanes** | Disinfectant by-products formed when chlorine reacts with organic compounds in water. These halogenated organics are named as derivatives of methane and include suspected carcinogens. |
| **ultrafiltration** | Pressure-driven membrane operations that use porous membranes for the removal of dissolved and colloidal matter. Ultrafiltration is commonly used to remove colloidal material and large molecules with molecular weights in excess of 5000. |
| **underflow** | The concentrated solids removed from the bottom of a tank or basin. |
| **upset** | An unexpected disturbance of a process or operation. |
| **urea** | A soluble nitrogen compound that is a component of mammalian urine. |
| **volatile suspended solids** | Organic content of suspended solids in a water or wastewater. Determined after a sample is heated to 550°C. |
| **waste activated sludge** | Excess activated sludge that is discharged from an activated sludge treatment process for further treatment and disposal. |
| **wastewater** | Liquid or waterborne wastes polluted or fouled from households or commercial or industrial operations, along with any surface water, stormwater or groundwater infiltration. |
| **wastewater treatment** | Process of improving the quality of wastewater. The term can refer to any part or all parts of the process by which raw wastewater is transformed through biological, biochemical and physical means to reduce contaminant concentrations to prescribed levels prior to release to the environment. A wastewater treatment process typically consists of primary, secondary and tertiary treatment. |
| **weir** | A baffle over which water flows. |
| **weir loading** | The rate of flow out of a basin, stated as the volume of liquid passing over a stated length of weir per unit of time. |

# Appendix D
# (Conversion Factors)

## LENGTH

| ft. | in. | m | cm |
|---|---|---|---|
| 1 | 12 | $3.048 \times 10^{-1}$ | 30.48 |
| 0.083 | 1 | $0.254 \times 10^{-1}$ | 2.54 |
| 3.281 | $3.937 \times 10^{1}$ | 1 | 100 |
| $3.281 \times 10^{-2}$ | $3.937 \times 10^{-1}$ | 0.01 | 1 |

## AREA

| ft.$^2$ | in.$^2$ | m$^2$ | cm$^2$ |
|---|---|---|---|
| 1 | 144 | $0.929 \times 10^{-1}$ | 929 |
| $6.944 \times 10^{-3}$ | 1 | $6.452 \times 10^{-4}$ | 6.452 |
| 10.764 | 1550 | 1 | $1.0 \times 10^{4}$ |
| $10.764 \times 10^{-4}$ | 0.155 | 1.0 × 10-4 | 1 |

## VOLUME

| U.S. gallon | ft.$^3$ | L | m$^3$ | imperial gallon |
|---|---|---|---|---|
| 1 | 0.134 | 3.785 | $3.785 \times 10^{-3}$ | $8.327 \times 10^{-1}$ |
| 7.481 | 1 | 28.317 | 0.028 | 6.229 |
| 0.264 | 0.035 | 1 | $1 \times 10^{-3}$ | $2.198 \times 10^{-1}$ |
| $2.642 \times 10^{2}$ | $3.531 \times 10^{1}$ | 1000 | 1 | $21.942 \times 10^{1}$ |
| 1.201 | 0.161 | 4.545 | $4.545 \times 10^{-3}$ | 1 |

## TIME

| d | h | min | s |
|---|---|---|---|
| 365 | 8760 | 525 600 | $31.536 \times 10^{6}$ |

## FLOW RATE

| mgd (U.S. gallon) | gpm (U.S. gallon) | $ft.^3/s$ | $ft.^3/min$ | L/s | $m^3/d$ | gpm (imperial gallon) |
|---|---|---|---|---|---|---|
| 1 | 694.444 | 1.547 | $9.282 \times 10^1$ | 43.747 | $0.378 \times 10^4$ | 577.750 |
| $0.144 \times 10^{-2}$ | 1 | $2.228 \times 10^{-3}$ | 0.134 | 0.063 | $0.545 \times 10^1$ | 0.833 |
| 0.646 | 448.831 | 1 | 60 | 28.317 | 2,446.589 | 374.308 |
| 0.011 | 7.481 | 0.017 | 1 | 0.472 | 40.781 | 6.239 |
| 0.023 | 15.851 | 0.035 | 2.119 | 1 | $8.641 \times 10^1$ | 13.221 |
| $0.264 \times 10^{-3}$ | 0.183 | $0.409 \times 10^{-3}$ | 0.025 | 0.012 | 1 | 0.153 |
| $1.729 \times 10^{-3}$ | 1.199 | $2.679 \times 10^{-3}$ | 0.164 | 0.079 | 6.549 | 1 |

## MASS

| ton | lb | kg | g |
|---|---|---|---|
| 1 | 2000 | 907.185 | 907 184.70 |
| $0.005 \times 10^{-1}$ | 1 | 0.454 | 454 |
| $1.10 \times 10^{-3}$ | 2.205 | 1 | 1000 |
| $1.10 \times 10^{-6}$ | $2.205 \times 10^{-3}$ | $10^{-3}$ | 1 |

## TEMPERATURE

| °F | °C | °K |
|---|---|---|
| °F | 5/9 (°F − 32) | 5/9 (°F + 255.38) |
| 9/5 °C + 32 | °C | °C + 273.16 |
| 9/5 °K − 459.69 | °K − 273.16 | °K |

## PRESSURE

| $lb./in.^2$ | ft. water | mm Hg | atm | $kg/cm^2$ | $N/m^2$ (Pa) |
|---|---|---|---|---|---|
| 1 | 2.307 | 2.036 | 0.068 | $7.031 \times 10^{-2}$ | $6.895 \times 10^3$ |
| 0.433 | 1 | 0.883 | $0.295 \times 10^{-1}$ | $0.305 \times 10^{-1}$ | $2.989 \times 10^3$ |
| $4.912 \times 10^{-1}$ | 1.133 | 1 | $3.342 \times 10^{-2}$ | $3.453 \times 10^{-2}$ | $3.377 \times 10^3$ |
| 14.70 | $3.393 \times 10^1$ | $2.992 \times 10^1$ | 1 | 1.033 | $1.013 \times 10^5$ |
| 14.225 | 32.783 | $2.896 \times 10^1$ | 0.968 | 1 | 98 070 |
| $1.450 \times 10^{-4}$ | $3.349 \times 10^{-4}$ | $2.961 \times 10^{-4}$ | $0.987 \times 10^{-5}$ | $0.102 \times 10^{-4}$ | 1 |

## POWER

| kW | BTU/min | Horsepower | ft.·lb./s |
|---|---|---|---|
| 1 | $5.692 \times 10^1$ | 1.341 | $7.376 \times 10^2$ |
| 0.018 | 1 | 0.024 | $1.297 \times 10^1$ |
| 0.746 | $4.244 \times 10^1$ | 1 | 550 |
| $1.356 \times 10^{-3}$ | 0.077 | 1.818 | 1 |

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